Part II

Seismic Evaluation Procedures Modified from the SQUG GIP

5. CAPACITY VERSUS DEMAND

5.1 INTRODUCTION

A screening guideline which should be satisfied to evaluate the seismic adequacy of an item of equipment identified in the Seismic Equipment List (SEL), as described in Chapter 4, is to confirm that the seismic capacity of the equipment is greater than or equal to the seismic demand imposed on it. This chapter addresses the determination of the seismic demand and capacity for the equipment as well as the comparison of the demand to the capacity. Note that a comparison of seismic capacity to seismic demand is also made in Chapter 6 for the equipment anchorage, in Chapter 9, Section 10.4.1, Section 10.4.3, and Section 10.5.1 for the equipment class evaluations using screening procedures, and in Chapter 11 for relays mounted in the equipment.

This chapter first presents the general description and techniques for computing the seismic demand and capacity, followed by the comparison of the demand to the capacity. In Section 5.2, the seismic demand is defined by the Seismic Demand Spectrum (SDS). The SDS is based on the Design Basis Earthquake (DBE) as defined for DOE facilities. The input motion for the equipment is determined by computing an in-structure response spectrum based on the DBE and the frequency response of the structure in which the equipment is mounted. Scaling factors are applied to the in-structure response spectrum to compute the SDS. In Section 5.3, the seismic capacity is represented by the Reference Spectrum, Generic Equipment Ruggedness Spectrum (GERS), or qualification test spectrum. Note that the Reference Spectrum and GERS can be used for representing seismic capacity of equipment only if the equipment meets the intent of the caveats for its equipment class as described in Chapter 8. Finally, in Section 5.4 the SDS is compared to the appropriate capacity spectrum.

The DOE Seismic Evaluation Procedure is intended primarily for systems and components identified as Performance Category (PC)-2 or higher. As discussed in DOE Orders and standards, the performance goal description for PC-1 is to maintain occupant safety during and/or immediately following an earthquake, while PC-2 and higher categories add goals such as continued operation with minimum interruption. Within the DOE graded approach, the primary concern for PC-1 structures is to prevent major structural damage or collapse that would endanger personnel. This concern is consistent with the goal of the model building codes, such as the Uniform Building Code (UBC) (Ref. 69), for general facilities to maintain life safety during earthquakes. The provisions of the UBC or similar building code should be followed for PC-1 systems and components since continued operation is not a requirement. For PC-2 and higher systems and components, the provisions of the DOE Seismic Evaluation Procedure satisfy the qualitative description of the performance goals for those categories and can be used to evaluate their capacity to at least have continued operation with minimum interruption during and/or immediately following an earthquake.

5.2 SEISMIC DEMAND

5.2.1 Design Basis Earthquake

For DOE facilities, the Design Basis Earthquake (DBE) is a specification of the mean seismic ground motion at the facility site for the earthquake-resistant design or evaluation of the structures, systems, and components at that site. The DBE is defined by ground motion parameters determined from mean seismic hazard curves and a design response spectrum shape. These hazard curves relate hazard exceedance probabilities to response quantities, such as peak ground acceleration. The methodologies for determining the seismic environment are described in DOE-STD-1022 (Ref. 70) and DOE-STD-1023 (Ref. 71). While DOE-STD-1022 provides procedures for site characterization, DOE-STD-1023 provides procedures for the development of hazard curves and spectra, such as the DBE, using parameters determined from the site characterization.

Many DOE sites have determined their site-specific DBE and have documented information about their DBE in Safety Analysis Reports (SARs) and reports in the hazards control or plant engineering departments of the DOE site.

As discussed in DOE-STD-1020 (Ref. 6), the preferable shape for the median deterministic DBE response spectrum should be site-specific and consistent with earthquake hazard parameters such as magnitudes, distances, and soil profiles. If a site-specific response spectrum shape is unavailable, a median standardized spectral shape may be used as long as the shape is consistent with or conservative for the site conditions. A recommended standardized spectral shape is shown in Figure 5.2-1, which is the shape defined in NUREG/CR-0098 (Ref. 72). The control points for the spectral shape in Figure 5.2-1 are provided in Table 5.2-1.

Table 5.2-1 Control Points for NUREG/CR-0098 Spectral Shape

Frequency (Hertz)	Spectral Acceleration (g)
0.1	$0.395\ d_{max} / g$
$v_{max} / (2 \pi d_{max})$	$v^2_{max} / (g\ d_{max})$
$(g\ a_{max}) / (2 \pi v_{max})$	a_{max}
8.0	a_{max}
33.0	a_g
100.0	a_g

where (for competent soil, $V_s < 3{,}500$ ft/sec, and for 50% spectra):

PGA - peak ground acceleration

β - percent damping

g = acceleration due to gravity (in/sec^2)

a_g = PGA (g)

v_g = $48\ a_g$ (in/sec)

d_g = $36\ a_g$ (in)

a_{max} = $a_g\ (3.21 - (0.68 \ln \beta))$

v_{max} = $v_g\ (2.31 - (0.41 \ln \beta))$

d_{max} = $d_g\ (1.82 - (0.27 \ln \beta))$

DOE-STD-1020 also discusses techniques for addressing the effective peak acceleration as compared to the predicted instrumental peak ground acceleration reported in some probabilistic seismic hazard assessments for sites at short epicentral distances. Typically, the effective peak acceleration is lower than the peak ground acceleration. While it is appropriate in seismic evaluations to remove sources of excessive conservatism, use of the effective peak acceleration for the evaluation of the functionality of active systems and components may not be conservative and should be peer reviewed on a site-specific basis. The effective peak acceleration may not be conservative because many types of active systems and components are relatively stiff and may no longer operate if the seismic demand requires inelastic response to the peak ground acceleration.

In order to demonstrate that DOE facilities are capable of resisting a specified level of seismic demand, it must be demonstrated that there is a sufficiently low probability of damage or failure of those facilities consistent with established performance goals as defined in DOE Orders and DOE-STD-1020. As discussed in Sections 1.2 and 4.1, the annual exceedance probability for a facility is determined by its performance category and the equipment in the SEL are classified into a particular performance category in accordance with DOE-STD-1021 (Ref. 7). Associated with each performance category is a different performance goal and an accompanying hazard exceedance probability which specifies the level of the DBE for each category.

DOE-STD-1020 permits some relief in the criteria for the seismic evaluation of systems and components in existing facilities. For existing facilities, the seismic evaluations may use a natural phenomena hazard exceedance probability that is twice the value specified for new facilities. This relief corresponds to a slight reduction (approximately 10-20%) in the seismic loads for the DBE. The basis of this slight reduction is contained in Reference 73. Use of the relief for specific existing facilities should follow the provisions in DOE-STD-1020.

The DBE is established at a higher annual frequency of occurrence than the acceptable annual probability of failure of the structures, systems, or components, so scale factors and experience data factors are required to achieve the appropriate risk reduction. These scale factors are similar to safety factors or the inherent conservatism in the acceptance criteria of structural design codes. The basis for the scale factors is provided in References 24 and 73 and the scale factors are shown in Table 5.2-2.

Table 5.2-2 Scale Factors

Performance Category[1]	Scale Factor (SF)
2	0.67
3	1.00
4	1.25

In the design of new equipment, rules are specified such that a known margin exists between the design value and the ultimate failure level. This margin has been considered in developing the provisions of DOE-STD-1020 as discussed in References 6, 24, and 73. A similar margin is required for the use of capacity obtained from experience data. Section 5.3 discusses the different types of capacity representation. The margin between the design and ultimate failure values are contained in the experience data factor, F_{ED}, defined in Reference 24 and shown in Table 5.2-3.

Table 5.2-3 Experience Data Factors

Capacity Representation[2]	F_{ED}
Reference Spectrum	1.0 SF
GERS	1.4 SF
Relay GERS	1.8 SF
Qualification Test	1.4 SF

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1 The Performance Category for each item of equipment in the SEL is determined using the provisions in Chapter 4 and DOE-STD-1021 (Ref. 7).

2 Definitions for the different capacity representations are provided in Sections 2.1.3.1 and 5.3.

5.2.2 In-Structure Response Spectrum[3]

For buildings, the DBE defines the seismic demand at the foundation of the structure. For equipment, the demand is defined in terms of the input motion applied at the appropriate attachment point(s) of the equipment. This demand or input motion is generally represented by an in-structure response spectrum (IRS). The IRS will differ significantly from the DBE spectrum because it is essentially filtered and / or amplified through the building. To use the provisions of the DOE Seismic Evaluation Procedure, the demand at the attachment point(s) of the equipment must consider the effects of structural filtering and / or amplification. Methods for determining the IRS with dynamic analyses are described in DOE-STD-1020 (Ref. 6) and ASCE 4 (Ref. 74). As discussed in ASCE 4, the IRS must account for uncertainties by spectral broadening or peak shifting. Additional guidance on computing IRS is provided in Sections 2.3 and C.4 of DOE-STD-1020. In DOE-STD-1020, dynamic analyses which may use IRS are only specified for PC-3 and PC-4 systems and components. In order to use the methodology in the DOE Seismic Evaluation Procedure, IRS should be developed as well for PC-2 systems and components in the SEL. Guidance for determining in-structure spectra for PC-2 systems and components is provided in the model building codes such as the UBC (Ref. 69) and the National Earthquake Hazards Reduction Program (NEHRP) Provisions (Ref. 75).

Realistic, median-centered in-structure response spectra are defined as response spectra which are based on realistic damping levels for the structure (including the effects of embedment and wave-scattering) and on structural dynamic analysis using realistic, best estimate modeling parameters and calculational methods such that no intentional conservatism enters into the process. These in-structure response spectra should be based on a ground response spectrum defined by the DBE as determined in DOE-STD-1023. For existing facilities with an approved Safety Analysis Report (SAR), the in-structure response spectra included in the SAR may be used as appropriate. Examples of realistic damping values are given in DOE-STD-1020 and EPRI Report NP-6041 (Ref. 18). The effects of embedment, wave scattering, and other soil-structure interaction (SSI) effects can be accounted for by using the methods in ASCE 4 by using frequency shifting rather than peak broadening. A spectral reduction factor can be used for considering the effects of horizontal spatial variation.

DOE-STD-1020 recommends the procedures in ASCE 4 for generating in-structure response spectra. The experience data factors, F_{ED}, listed in Table 5.2-3 are appropriate when the in-structure response spectra are generated in accordance with DOE-STD-1020. In some cases, in-structure response spectra may be developed with varying conservatism which is different than that defined in DOE-STD-1020. Reference 24 outlines methods to account for variation in the determination of in-structure response spectra with different levels of conservatism. The Seismic Safety Margins Research Program (Ref. 57 and 58) has demonstrated the large conservatism which exists in traditionally-computed, conservative design in-structure response spectra versus realistic, median-centered in-structure response spectra. The specific assumptions made in generating in-structure response curves should be reviewed by SCEs using the guidance provided in Appendix A of Reference 19.

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3 Based on Section 4.2.4 of SQUG GIP (Ref. 1)

5.2.3 Seismic Demand Spectrum

To evaluate the seismic demand at the attachment point(s) of equipment, an in-structure response spectrum (IRS) is scaled by F_{ED} to determine the Seismic Demand Spectrum (SDS) according to the following equation:

$$SDS = F_{ED} \times IRS$$

where:

SDS - Seismic Demand Spectrum or Scaled In-Structure Response Spectrum. For relays, the SDS is modified to account for in-cabinet amplification. Chapter 11 provides two methods for modifying the SDS for relays mounted in cabinets.

F_{ED} - Experience Data Factor. It depends on the performance category and capacity representation of the equipment and is defined in Tables 5.2-2 and 5.2-3.

IRS - In-Structure Response Spectrum. It is determined for the appropriate attachment point(s) of the equipment and is a function of the DBE for the facility and the frequency content of the structure supporting the equipment.

Additional information on techniques for computing the seismic demand spectrum are provided in Step 1 of Section 6.4.2. In this section, an approximate technique for scaling seismic demand spectra, which are defined for different damping values, is discussed.

5.2.4 Total Demand

The total demand (D_{TI}) is a combination of seismic loads (D_{SI}) and concurrent non-seismic loads (D_{NS}).

$$D_{TI} = D_{SI} + D_{NS}$$

where:

D_{TI} - Total Demand

D_{SI} - Seismic Loads. According to DOE-STD-1020 (Ref. 6), the dynamic analyses used to compute the seismic loads for PC-3 and PC-4 systems and components must consider all three orthogonal components of earthquake ground motion (two horizontal and one vertical). In order to use the methodology in the DOE Seismic Evaluation Procedure, all three orthogonal components of earthquake ground motion should be considered for PC-2, PC-3, and PC-4 systems and components. The earthquake ground motion is described by the SDS defined in Section 5.2.3. For near-field sites, the vertical component of the DBE may exceed the horizontal components. Responses from the various directional components should be combined with acceptable combinations techniques, such as the Square-Root-Sum-of-the-Squares (SRSS) and the 100-40-40-Rule, in accordance with ASCE 4 (Ref. 74).

D_{NS} - Non-Seismic Operational Loads

When comparing D_{TI} to seismic capacity based on earthquake experience data as defined in Section 5.3.1 or generic seismic testing data as defined in Section 5.3.2, the effects of all three orthogonal components of the earthquake ground motion and the effects of non-seismic operational loads are typically not explicitly considered for equipment adequacy assessment as described below:

(a) According to Section 4.2.3 of the SQUG GIP (Ref. 1), the vertical component of the ground response spectrum is not explicitly considered for equipment adequacy assessment. In general, it is considered that equipment is more sensitive to horizontal motion than vertical motion. Evaluation of the effects of the vertical component is implicit in the horizontal motion assessment since the earthquake-experience facilities typically experienced relatively higher vertical motion than that explicitly considered. When using GERS, the generic seismic testing included effects of vertical motion which was consistent with that explicitly considered.

(b) Equipment in the earthquake-experience database was subjected to non-seismic operating loads concurrent with the seismic loads. In many cases, the non-seismic loads were implicitly included along with the horizontal seismic loads and in defining the caveats for the Reference Spectrum. Note that there may be facility-specific equipment that is subjected to operating loads which were not implicitly included in the experience database. For equipment subjected to both operating and seismic loads, the database may need to be reviewed to determine if the operating loads were implicitly considered. If the operating loads were not implicitly considered, then their effects should be considered concurrently with the seismic loads.

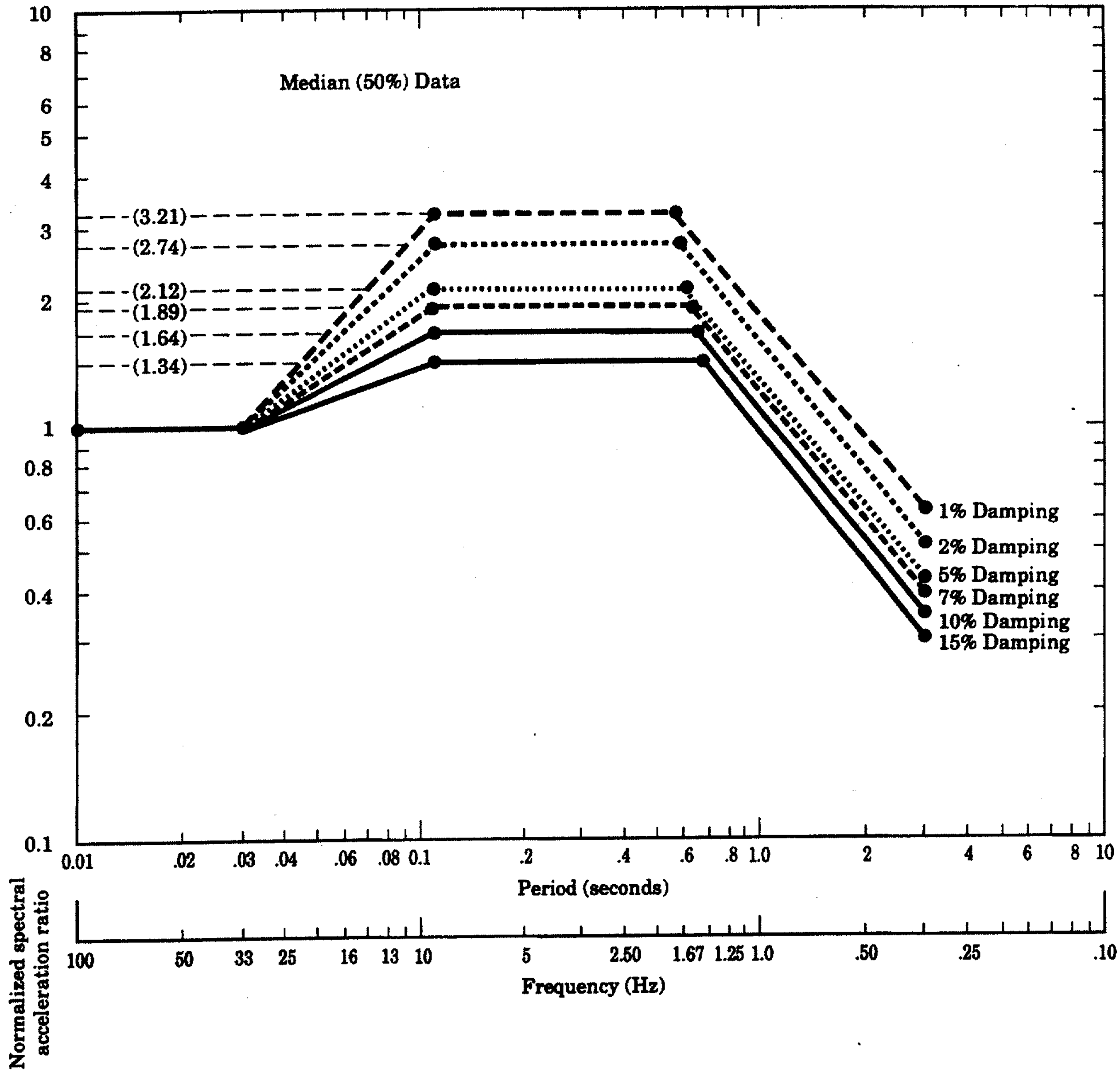

Figure 5.2-1 Median Standardized Spectral Shape Defined in NUREG/CR-0098, the Newmark and Hall 50% Exceedance Horizontal Spectra for Soil (References 72 and 76)

5.3 EQUIPMENT CAPACITY

5.3.1 Seismic Capacity Based on Earthquake Experience Data[4]

Earthquake experience data was obtained by surveying and cataloging the effects of strong ground motion earthquakes on various classes of equipment mounted in conventional power plants and other industrial facilities. The results of this effort are summarized in Reference 35. Based on this work, a Reference Spectrum was developed which represents the seismic capacity of equipment in the earthquake experience equipment class. A detailed description of the derivation and use of this Reference Spectrum is contained in Reference 19 and this reference should be reviewed by the SCEs before using the Reference Spectrum. The Reference Spectrum and the four spectra from which it is derived are shown in Figure 5.3-1. Figure 5.3-2 shows the Reference Spectrum and its defining response levels and frequencies.

The Reference Spectrum can be used to represent the seismic capacity of equipment in a DOE facility when this equipment is determined to have characteristics similar to the earthquake experience equipment class and meets the intent of the caveats for that class of equipment as defined in Chapter 8. Use of the Reference Spectrum for comparison with the Seismic Demand Spectrum (SDS) is described in Section 5.4.

5.3.2 Seismic Capacity Based on Generic Seismic Testing Data[5]

A large amount of data was collected from seismic qualification testing of nuclear power plant equipment. This data was used to establish a generic ruggedness level for various equipment classes in the form of Generic Equipment Ruggedness Spectra (GERS). The development of the GERS and the limitations on their use (caveats) are documented in Reference 40. Copies of the non-relay GERS along with a summary of the caveats to be used with them are included in Chapter 8. A copy of a relay GERS is included in Chapter 11. SCEs should review Reference 40 to understand the basis for the GERS.

GERS can be used to represent the seismic capacity of an item of equipment in a DOE facility when this equipment is determined to have characteristics which are similar to the generic testing equipment class and meets the intent of the caveats for that class of equipment as defined in Chapter 8. Use of the GERS for comparison with the Seismic Demand Spectrum (SDS) is described in Section 5.4.

5.3.3 Equipment-Specific Seismic Qualification

Equipment-specific seismic qualification techniques, as used in newer DOE facilities, may be used instead of the methods given in Section 5.3.1 and 5.3.2. With this technique, shake-table tests should be performed in accordance with IEEE-344-75 Standards (Ref. 12) or more current standards.

Equipment-specific seismic qualification can be useful for equipment classes discussed in Chapter 10. Some of these equipment classes do not have the Reference Spectrum or GERS to define their capacity. With seismic qualification techniques, a test spectrum can be generated for these classes of equipment and this spectrum must be scaled with the F_{ED} for Qualification Test in Table 5.2-3.

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[4] Based on Section 4.2.1 of SQUG GIP (Ref. 1)

[5] Based on Section 4.2.2 of SQUG GIP (Ref. 1)

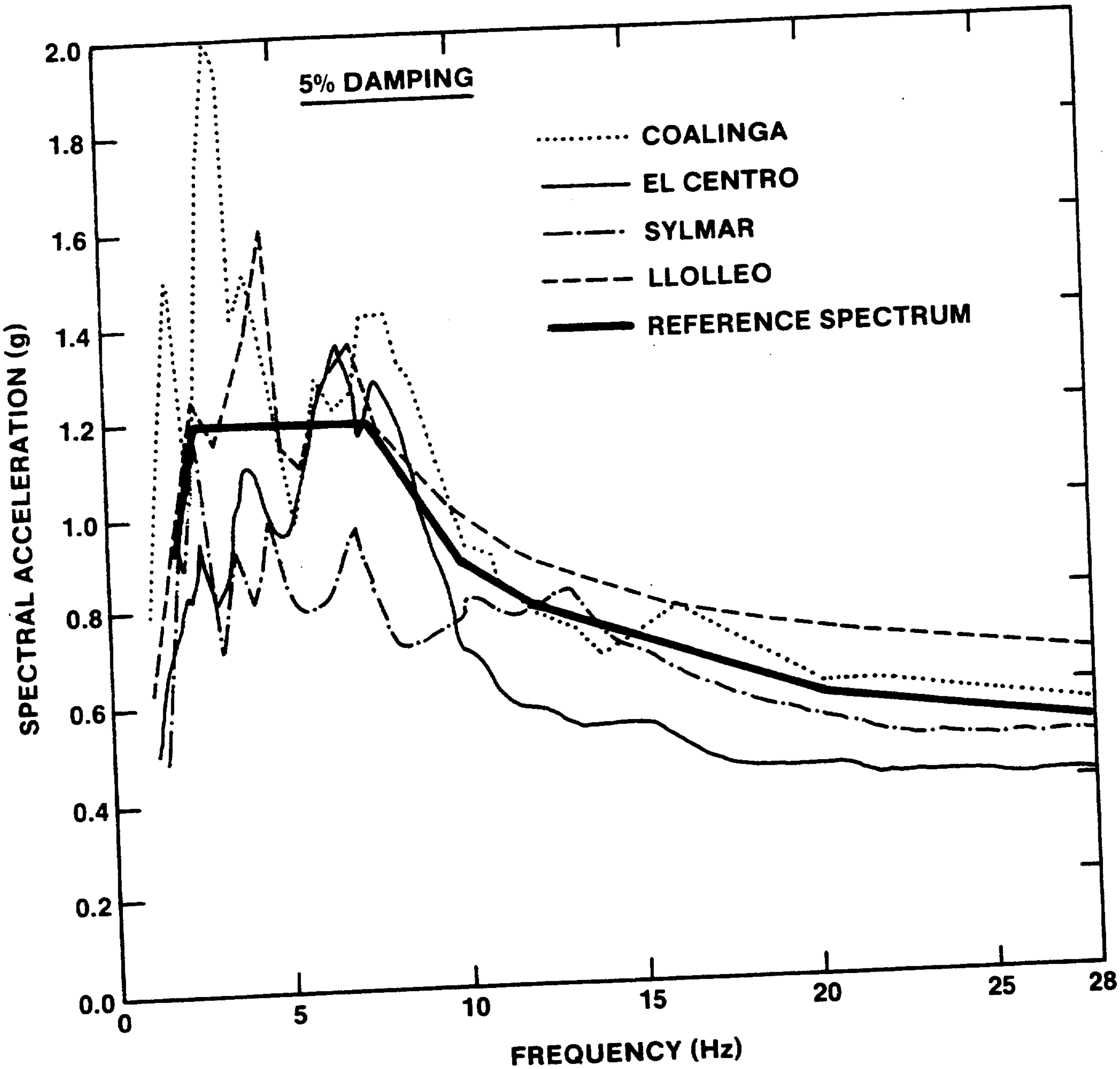
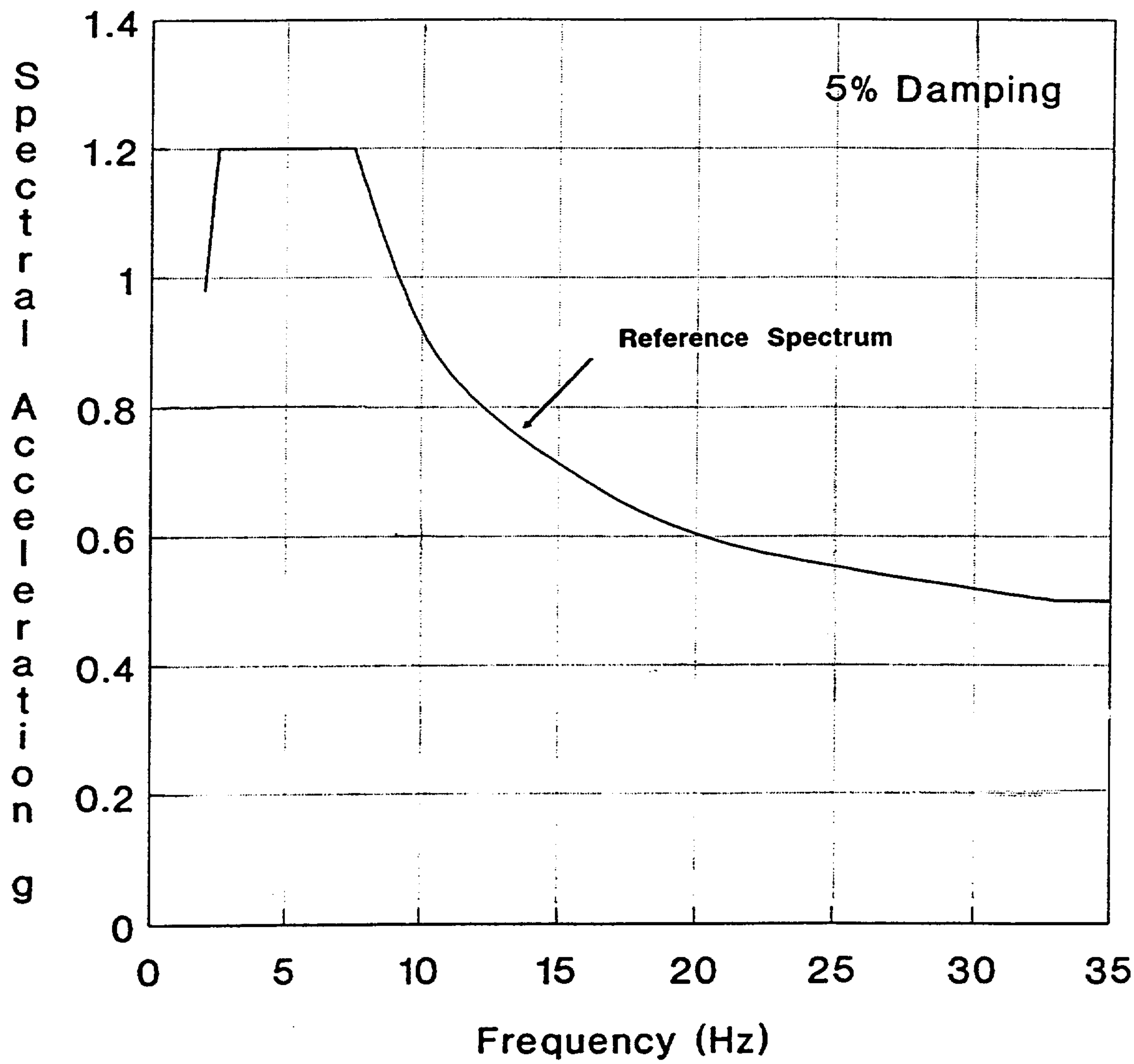


Figure 5.3-1 Horizontal Response Spectra Representing the Earthquake Experience Database (Reference 19)



Frequency (Hz)	2.0	2.5	7.5	8.0	10	12	16	20	28	33
Spectral Acceleration (g)	.98	1.2	1.2	1.13	.90	.80	.68	.59	.53	.50

Figure 5.3-2 Seismic Capacity Represented by Reference Spectrum Based on Earthquake Experience Database (Figure 4-2 of SQUG GIP, Reference 1)

5.4 EXPERIENCE-BASED CAPACITY COMPARED TO SEISMIC DEMAND

This section addresses the comparison of experience-based seismic capacity to seismic demand for the equipment. The seismic capacity of equipment can be represented by a Reference Spectrum based on earthquake experience data, a Generic Ruggedness Spectrum (GERS) based on generic seismic test data, or a test spectrum from equipment-specific seismic qualification as respectively described in Sections 5.3.1, 5.3.2, and 5.3.3. Note that the first two methods of representing seismic capacity of equipment can only be used if the equipment meets the intent of the caveats for its equipment class as described in Chapter 8. The seismic capacity of an item of equipment is compared to its seismic demand which is defined in terms of an in-structure response spectrum (IRS). As discussed in Section 5.2, the IRS is scaled with the applicable scale factors to determine the Seismic Demand Spectrum (SDS).

5.4.1 Comparison of Equipment Seismic Capacity to Seismic Demand[6]

An in-structure response spectrum can be used for comparison to Reference Spectrum, GERS, or test spectrum for equipment which is mounted at any elevation in the facility and/or for equipment with any natural frequency. The Reference Spectrum, GERS, or test spectrum are used to represent the capacity of the equipment. The SDS associated with the DBE for a DOE facility can be used to represent the seismic demand applied to the facility equipment. One of the following comparisons of capacity and demand, as illustrated in Figure 5.4-1, is made:

- Reference Spectrum envelops the Seismic Demand Spectrum (SDS)

  Reference Spectrum (Section 5.3.1) $\geq$ SDS

- GERS envelops the Seismic Demand Spectrum (SDS)

  GERS (Section 5.3.2) $\geq$ SDS

- Test spectrum envelops the Seismic Demand Spectrum (SDS)

  Seismic Qualification Tests (Section 5.3.3) $\geq$ SDS

- Relay GERS envelops the In-cabinet Demand Spectrum (IDS). Section 11.3 discusses techniques for calculating the IDS using the Seismic Demand Spectrum (SDS).

  Relay GERS (Section 11.2) $\geq$ IDS

For these comparisons, the largest horizontal component of the 5% damped in-structure response spectra is used for the location in the facility where the item of equipment is mounted. An approximate technique for scaling in-structure response spectra by their damping ratios is provided in Section 6.4. The in-structure response spectrum used for the seismic demand should be representative of the elevation in the structure where the equipment is anchored and receives its seismic input. This elevation should be determined by the SCEs during the facility walkdown. If one of the comparisons shown above is not satisfied, then the equipment being evaluated is an outlier. Methods for resolving outliers are provided in Chapter 12.

6 Based on Section 4.2.4 of SQUG GIP (Ref. 1)

5.4.2 Enveloping of Seismic Demand Spectrum[7]

To evaluate seismic adequacy, in general, the seismic capacity spectrum should envelop the SDS over the entire frequency range of interest (typically 1 to 33 Hz). There are two special exceptions to this general rule:

- The seismic capacity spectrum needs only to envelop the SDS for frequencies at and above the conservatively estimated lowest natural frequency of the item of equipment being evaluated.

  Caution should be exercised when using this exception because an equipment assembly (e.g., electrical cabinet lineup) may consist of many subassemblies, each manifesting its fundamental mode of vibration at different frequencies. The lowest natural frequency of each subassembly should be determined with high confidence using the guidance provided below in Section 5.4.3. It is noted that unless the equipment is tested with a high-level vibratory input, the fundamental frequency can be difficult to estimate, especially for complex structural equipment.

- Narrow peaks in the SDS may exceed the seismic capacity response spectrum if the average ratio of the SDS to the capacity spectrum does not exceed unity when computed over a frequency range of 10% of the peak frequency (e.g., 0.8 Hz range at 8 Hz). Note that it is permissible to use unbroadened SDS for this comparison, however when doing so, uncertainty in the natural frequency of the building structure should be addressed by shifting the frequency of the SDS at these peaks. An acceptable method of peak shifting is described in ASCE 4 (Ref. 74). A reference or basis for establishing the degree of uncertainty in the natural frequency of the building structure should be included in the facility-specific seismic evaluation records.

If either of these exceptions are used, the Screening Evaluation Work Sheets (SEWS) should be marked to indicate the exception that has been invoked.

5.4.3 Lowest Natural Frequency[8]

When it is necessary to determine the lowest natural frequency of an item of equipment, the SCEs may, in most cases, estimate a lower bound for this frequency based on their experience, judgment, and available data. Methods for frequency estimation are provided in Reference 77. The lowest natural frequency of concern is that of the lowest natural mode of vibration that could adversely affect the safety function of the equipment. The modes of vibration which should be considered are:

- The overall structural modes of the equipment itself and

- The modes for internal structures (e.g., flexural mode for door panels) which support components needed to accomplish the safety function of the equipment.

- The modes of devices which are needed to accomplish the safety function of the equipment. A value of 5 Hertz is recommended and higher values should be appropriately justified.

In addition, the SCEs should also be alert and note any items of concern within the "box" which could be seismically vulnerable. This would include components mounted in the "box" which have known low natural frequencies, seismic vulnerabilities, or improper mounting (e.g., loose or

[7] Based on Section 4.2 of SQUG GIP (Ref. 1)
[8] Based on Section 4.2 of SQUG GIP (Ref. 1)

missing bolts). If these types of situations are found during the seismic review, their presence may constitute a third type of vibrational mode and their influence should be included in the estimate of the lowest natural frequency and the assessment of the seismic adequacy of the equipment.

5.4.4 Guidance for Evaluating In-Line Equipment[9]

The amplified response of in-line equipment which is supported by piping (e.g., valves, valve operators, and sensors) is handled differently when using the Reference Spectrum or the GERS as the seismic capacity of the equipment. When using the Reference Spectrum, it is not necessary to account for amplification of the piping system between the anchor point of the piping system (i.e., the floor or wall of the building) and the point on the piping system where the item of equipment is attached. This is because the effect of amplified response in piping systems is accounted for in the earthquake experience data base.

When using GERS as the seismic capacity of equipment, piping system amplifications should be accounted for when establishing the seismic demand on the in-line item of equipment. The amplification factor can be obtained from a dynamic piping analysis if one is available. As an alternative, the amplification factor may be estimated using judgment with peer review.

[9] Based on Section 4.2.4 of SQUG GIP (Ref. 1)

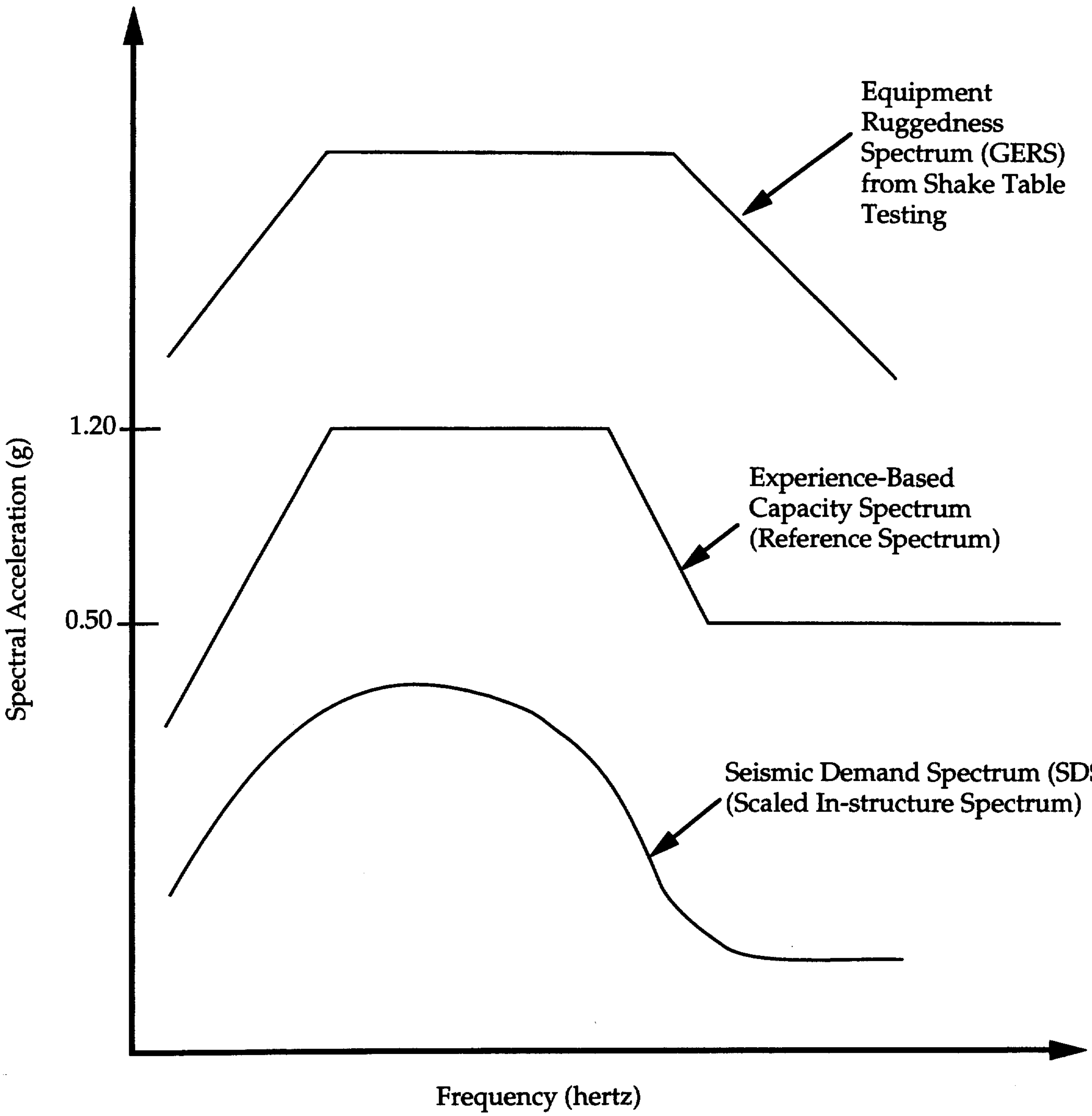


Figure 5.4-1 Comparison of Seismic Capacity Spectrum to Seismic Demand Spectrum

6. ANCHORAGE DATA AND EVALUATION PROCEDURE

6.1 INTRODUCTION[1]

The purpose of this chapter is to:

- Provide a general description of the anchorage evaluation procedure,
- Provide generic information on the various equipment classes for use in anchorage evaluations,
- Provide nominal allowable capacities for certain types of anchors, and
- Describe anchor-specific inspection checks and capacity reduction factors.

The four main steps for evaluating the seismic adequacy of equipment anchorage include:

1. Anchorage Installation Inspection (Section 6.2)
2. Anchorage Capacity Determination (Section 6.3)
3. Seismic Demand Determination (Section 6.4)
4. Comparison of Capacity to Demand (Section 6.5)

This chapter is organized with an evaluation of the installation adequacy and attributes of the anchorage given first. Next, the anchorage capacity is determined in Sections 6.3.1 to 6.3.9 and the steps in the capacity determination are grouped by the following anchor types:

Expansion Anchors

Cast-In-Place Bolts and Headed Studs

Cast-In-Place J-Bolts

Grouted-In-Place Bolts

The following two other anchor types are evaluated using separate procedures in Section 6.3.10:

Welds to Embedded Steel or Exposed Steel

Lead Cinch Anchors

Section 6.3 contains the main steps in the procedure for evaluating the seismic capacity of equipment anchorage. The sections contain a table of nominal allowable load capacities along with anchor-specific inspections which should be performed. In some cases a capacity reduction factor is given which may be used to lower the nominal allowable load capacities if the inspection check reveals that the installation does not meet the minimum guidelines.

Section 6.4 contains generic equipment characteristics for anchorage demand evaluations for use when equipment-specific data is not available for equipment mass, natural frequency, or damping. In addition, an approximate technique for scaling in-structure response spectra by their damping ratios is provided.

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1 Based on Sections C-Introduction and 4.4 of SQUG GIP (Ref. 1)

The material in this chapter is based on the information contained in Reference 41. The SCEs should not use the material contained in this chapter unless they have thoroughly reviewed and understood Reference 41.

Adequate anchorage is almost always essential to the survivability of an item of equipment. Lack of anchorage or inadequate anchorage has been a significant cause of equipment failing to function properly during and following past earthquakes. The screening approach for evaluating the seismic adequacy of equipment anchorage is based upon a combination of inspections, analyses, and engineering judgment. Inspections consist of measurements and visual evaluations of the equipment and its anchorage; supplemented by use of facility documentation and drawings. Analyses should be performed to compare the anchorage capacity to the seismic loadings (demand) imposed upon the anchorage. These analyses should be done using the guidelines contained in this chapter. Engineering judgment is also an important element in the evaluation of equipment anchorage. Guidance for making judgments is included, where appropriate, in this chapter and in anchorage reference documents.

There are various combinations of inspections, analyses, and engineering judgment which can be used to evaluate the adequacy of equipment anchorage. The SCEs should select the appropriate combination of elements for each anchorage installation based on the information available. For example, a simple hand calculation may be sufficient for a pump which has only a few, very rugged, anchor bolts in a symmetrical pattern. On the other hand, at times it may be advisable to use computer codes to determine the loads applied to a multi-cabinet motor control center if its anchorage is not symmetrically located. Likewise a trade-off can be made between the level of inspection performed and the factor of safety used for expansion anchor bolts. These types of trade-offs and others are discussed in this chapter.

This chapter describes the main steps for evaluating the seismic adequacy of anchorage. In some cases, specific inspection checks and evaluations apply to only certain types of anchors. Section 13.2 describes Screening and Evaluation Work Sheets (SEWS) which can be used as checklists to evaluate that all the appropriate steps in the anchorage evaluation procedure have been completed.

It is not necessary to perform an anchorage evaluation for in-line valves which are discussed in Sections 8.2.1 and 8.2.2. Likewise temperature sensors, which are discussed in Section 8.1.10, are relatively light, normally attached to another piece of equipment, and do not need an anchorage evaluation.

6.2 ANCHORAGE INSTALLATION INSPECTION

6.2.1 Installation Adequacy and Attributes[2]

To evaluate the seismic adequacy of anchorage, the anchorage installation and its connection to the base of the equipment should be checked. This inspection consists of visual checks and measurements along with a review of facility documentation and drawings where necessary. All accessible anchorage should be visually inspected. All practicable means should be tried to inspect inaccessible anchorage or those obstructed from view if they are needed for strength to secure the item of equipment or if they secure equipment housing essential relays (to avoid impact or excessive cabinet motion). For example, it is not considered practicable to resort to equipment disassembly or removal to inspect inaccessible anchorage. The basis for the engineering judgment for not performing these inspections should be documented.

[2] Based on Section 4.4.1 of SQUG GIP (Ref. 1)

Several general installation checks should be made of the anchorage. For welds, a visual check of the adequacy of the welded joint should be performed. For bolt or stud installation, a visual check should be made to determine whether the bolt or nut is in place and uses a washer where necessary. Oversized washers or reinforcing plates are recommended for thin equipment bases. Lock washers are recommended where even low-level vibration exists. For expansion anchors, a tightness check should be made to detect gross installation defects (such as oversized concrete holes, total lack of preload, loose nuts, damaged subsurface concrete, and missing plug for shell types) which would leave the anchor loose in the hole. The checks to be made on expansion anchors are discussed in detail in Section 6.3.9.

A check of the following equipment anchorage attributes should be made:

- Equipment Characteristics (i.e., estimation of mass, center of gravity location, natural frequency, damping, and equipment base overturning moment center of rotation) (see Section 6.4.1)
- Type of Anchorage (see Sections 6.3.1 and 6.3.2)
- Size and Location of Anchorage (see Section 6.2.2)
- Equipment Base Stiffness and Prying Action (see Section 6.2.4)
- Equipment Base Strength and Structural Load Path (see Section 6.2.5)
- Embedment Steel and Pads (see Section 6.2.6)
- Embedment Length (see Section 6.3.3)
- Gap at Threaded Anchors (see Section 6.2.3)
- Spacing Between Anchors (see Section 6.3.4)
- Edge Distance (see Section 6.3.5)
- Concrete Strength and Condition (see Section 6.3.6)
- Concrete Crack Locations and Sizes (see Section 6.3.7)
- Essential Relays in Cabinets (see Section 6.3.8)
- Installation Adequacy (see Section 6.3.9.1)

Not all of these attributes are applicable to all types of anchors. General guidelines for performing the checks are provided in the sections provided in the list. Engineering judgment should be exercised when making these checks. For example, it is not necessary to measure the spacing between anchor bolts if it is obvious they are much farther apart than the minimum spacing guidelines.

6.2.2 Size and Location of Anchorage[3]

The size of the anchors and the locations where they secure the item of equipment to the floor or wall are key parameters for establishing the capacity of the anchorage for that item of equipment. The nominal allowable capacities are listed according to the diameter of the anchor. Diameter is also used as a key parameter for defining the minimum embedment length, spacing between anchors, and edge distance. The number and location of the anchors which secure an item of equipment determine how the seismic loadings are distributed among all the anchors. Note that the nominal allowable capacities also apply to anchors in the tension zone of concrete; e.g., on the ceiling. Anchors in damp areas or harsh environments should be checked for corrosion deterioration if heavy surface rust is observed.

6.2.3 Gap at Threaded Anchors[4]

The size of the gap between the base of the equipment and the surface of the concrete should be less than about 1/4 inch in the vicinity of the anchors (as illustrated in Figure 6.2-1). This limitation is necessary to prevent excessive flexural stresses in the anchor bolt or stud and excessive bending moments on the concrete anchorage when shear loads are applied. Expansion anchors may have low resistance to imposed bolt bending moment which might result from gaps between base and floor. Anchorage with gaps larger than about 1/4 inch should be classified as outliers and evaluated in more detail. Guidance on resolving anchorage outliers is provided in Reference 78.

There should be no gap at the bolt or stud anchor locations for equipment containing essential relays. Gaps beneath the base of this equipment are not allowed since they have the potential for opening and closing during earthquake load reversals. This may create high frequency impact loadings on the equipment and any essential relays mounted therein could chatter.

The gap size can be checked by performing a visual inspection; a detailed measurement of the gap size is not necessary. The check for the presence of essential relays in equipment can be done in conjunction with the Relay Functionality Review described in Chapter 11.

6.2.4 Base Stiffness and Prying Action[5]

The base and anchorage load path of the equipment should be inspected to confirm that there is adequate stiffness and there is no significant prying action applied to the anchors. One special case of base flexibility is base vibration isolation systems. Guidelines for evaluating base vibration isolators are included at the end of this section.

There are two main concerns with the lack of adequate stiffness in the anchorage and load path. First, the natural frequency of the item of equipment could be lowered into the frequency range where dynamic earthquake loadings are higher. Second, the cabinet could lift up off the floor during an earthquake resulting in high frequency impact loadings on the equipment, and any essential relays mounted therein could chatter.

Prying action can result from eccentric loads within the equipment itself and between the equipment and the anchors. The concern is that these prying actions can result in a lack of adequate stiffness and strength and in additional moment loadings within the equipment or on the anchors.

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[3] Based on Section 4.4.1 - Check 3 of SQUG GIP (Ref. 1)
[4] Based on Section 4.4.1 - Check 6 of SQUG GIP (Ref. 1)
[5] Based on Section 4.4.1 - Check 12 of SQUG GIP (Ref. 1)

Thin framing members and clip angles may lack the strength and stiffness required to transfer loads to anchor bolts. Stiff load paths with little eccentricity are preferable for anchorage. Equipment constructed of sheet metal, such as motor control centers, switchgear, and instrumentation and control cabinets, is susceptible to these effects and should be carefully inspected for lack of stiffness and prying action. Figure 6.2-2 shows examples of stiff and excessively flexible anchorage connections with prying action. In Example "A" of this figure, the thin sheet metal may easily bend during uplift of the cabinet.

This unacceptable condition may be corrected by welding the outside edge of the cabinet base to the embedded steel as shown in Example "C". Care should be taken during welding to avoid burning through the thin sheet metal frame of the cabinet. Example "B" shows a thin sheet metal base which can also easily bend during uplift. This unacceptable condition may be corrected by adding a thick metal plate under the nut of the anchor bolt so that the effective thickness and size of the base is similar to the bottom leg of the structural angle shown in Example "D". Note that the prying effect of the eccentric load on the anchor bolt in Example "D" should be considered. Likewise, if the weld in Example "C" is actually nearer the edge of the embedded plate rather than at the center as shown, then prying and/or bending will be present in the embedded plate. Thin cabinet bases should be reinforced with angle framing so that seismic loads may be transferred to anchor points. In addition, oversized washers are required when anchors are bolted directly through thin sheet metal bases.

Heavy components that are mounted on upright channel sections may rely on weak-way bending of the channel to transfer shear loads to the anchorage. Unstiffened, light-gage channels may not have sufficient strength to handle this load transfer.

The checks for adequate stiffness and lack of prying action require considerable engineering judgment and can be done by a visual inspection of the anchorage installation. SCEs should also review by visual inspection the entire anchorage load path of the equipment for adequate stiffness. If the base is flexible or if prying action could occur, then the SCEs should exercise their judgment to lower the capacity of the anchorage accordingly.

If the equipment is mounted on a base vibration isolation system, then the isolators should be evaluated for seismic adequacy using the following guidelines. Base vibration isolators are vulnerable to failure during an earthquake for several reasons. Vibration isolators consisting primarily of one or several springs have failed during earthquakes when the springs could not resist lateral loads. Isolators manufactured of cast iron can shatter when subjected to earthquakes. Rubber and elastomer products in isolators can fail when bonding adhesives or the material itself fails. Other isolators have steel sections surrounding the spring element which at first appear stout; however, detailed review can reveal that seismic loads may be carried through small fillet or tack welds and through flat bearing plates which bend along their weak axis.

For a base vibration isolator system to be acceptable for seismic loads, the isolator system should have a complete set of bumpers to prevent excessive lateral movement in all directions. The bumpers should not only prevent any excessive lateral movement and torsion, but a positive method of resisting uplift should also be provided other than the springs themselves, or the rubber or adhesives in tension. If the bumpers do not have elastomeric pads to prevent hard impact, the effect of that impact on the equipment should be evaluated. (Note: Essential relays should not be mounted in such equipment.) Isolators which were specifically designed for seismic applications (not cast iron, unbraced springs, weak elastomers, etc.) may be accepted, provided suitable check calculations determine that all possible load combinations and eccentricities within the isolator itself, including possible impact loads, can be taken by the isolator system.

6.2.5 Equipment Base Strength and Structural Load Path[6]

The equipment base and structural load path should be checked to confirm that it has adequate strength, stiffness, and ductility to transmit the seismic loads from the center of gravity of the equipment to the anchorage. Several connections and support members may need to be checked in the evaluation to confirm that the weak link in the load path is addressed, e.g., the channel or stud embedment, the weld between the embedded steel and the cabinet base, and the connection bolts between the base of the equipment and its frame members. Friction connections, such as holddown clips, often pry off or completely slip out-of-place during seismic loading and become completely ineffective. Adequate anchorage requires positive connection.

This check should include such items as whether a washer is present under the nut or the head of the bolt, and if not present, whether one is necessary. A washer is not necessary if the base of the equipment is at least as thick as a standard washer with a hole no larger than the hole in a standard washer. Another item to check is whether the internal bolting and welds near the base of the equipment can carry the anchorage loads.

One example of inadequate strength in the equipment base was demonstrated during a shake table test of a motor control center in which all four corners of the assembly broke loose. The weld between the base channel and the shake table remained intact; however, the small 5/16-inch bolted connections between the base channel and the frame of the assembly broke. The check for adequate strength in the equipment base can be done by a visual inspection of the anchorage installation. This check should be done in conjunction with Section 6.2.4.

6.2.6 Embedment Steel and Pads[7]

If an item of equipment is welded to embedded steel or it is mounted on a grout pad or a large concrete pad, the adequacy of the embedded steel, the grout pad, or the large concrete pad should be evaluated.

Welds made to embedded steel transmit the anchor load to the embedment. The location of the weld should be such that large eccentric loads are not applied to the embedded steel. With welded anchors, the presence of weld burn-through in light-gage steel may indicate a weak connection. In addition, line welds have minimal resistance to bending moments applied about the axis of the weld. These moments may occur when there is weld only on one side of a flange. Puddle welds and plug welds used to fill bolt holes in equipment bases have relatively little capacity for applied tensile loads. Fillet welds built up across stacked shim plates may appear to be large but have very small effective throat area and thus low capacity.

If the embedment uses headed studs, the strength criteria should be used together with the generic guidelines contained in this section. Other types of cast-in-place embedments are not covered in this procedure and should be classified as outliers. The holding strength of these other types of embedments may be determined during the outlier evaluation by testing or by application of generally accepted engineering principles. Engineering judgment should be exercised to establish a conservative estimate of the concrete failure surface for outlier resolution of these other types of embedments. Manufacturer's test information or facility specific test information may be used in the outlier resolution of other types of embedments as appropriate. Factors of safety consistent with this procedure should be followed. Guidance on resolving anchorage outliers is provided in Reference 78.

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6 Based on Section 4.4.1 - Check 13 of SQUG GIP (Ref. 1)

7 Based on Section 4.4.1 - Check 14 of SQUG GIP (Ref. 1)

Equipment mounted on grout pads should be checked to confirm that the anchorage penetrate through the grout pad into the structural concrete beneath. Anchorage installed only in the grout pad have failed in past earthquakes and do not have the capacity values assigned to anchors in structural concrete. Inadequate embedment may result from use of shims or tall grout pads.

If an item of equipment is anchored to a large concrete pad, the pad should have reinforcing steel and be of sound construction (i.e., no prominent cracks). The pad/floor interface should also be evaluated to determine whether it can transmit the earthquake loads. For example, if there are sufficient reinforcement bars connecting the floor to the pad, then the connection is adequate. Also, if a chemical bonding agent was used between the pad and floor, the adhesion strength can typically develop the same strength as the concrete in tension and shear.

If there are no reinforcement bars or chemical bond between the pad and the floor, then the interface can typically resist only shear loadings (if the interface had been roughened at the time the pad was poured). It may be possible, in this case, to show that there are no tensile loads on the pad/floor interface due to either: (1) the center of gravity of the item of equipment being low, or (2) the weight of the pad itself acting as a ballast to resist the overturning moment. The adequacy check of the embedded steel, grout pad, and large concrete pad can be done with a visual inspection together with measurements and the use of drawings and other documents where necessary.

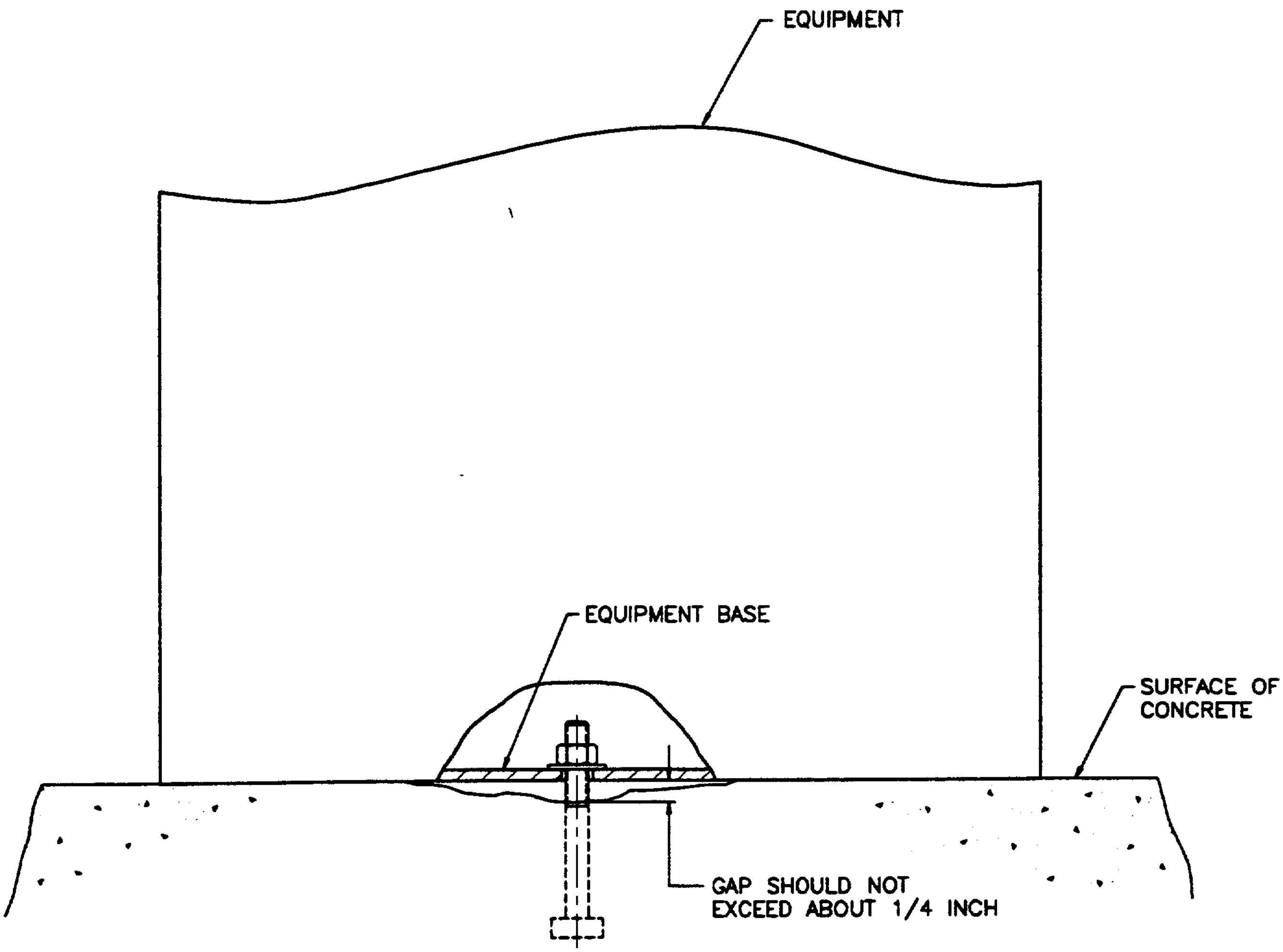
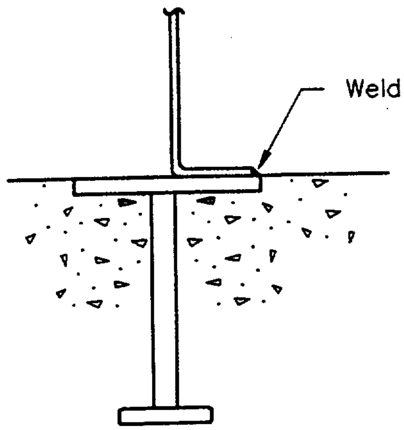
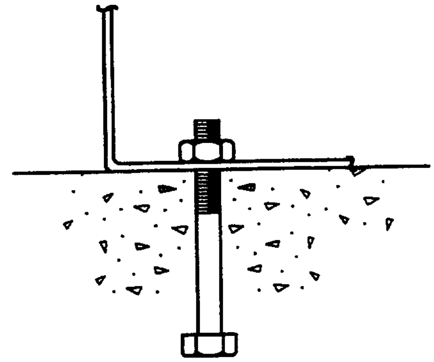


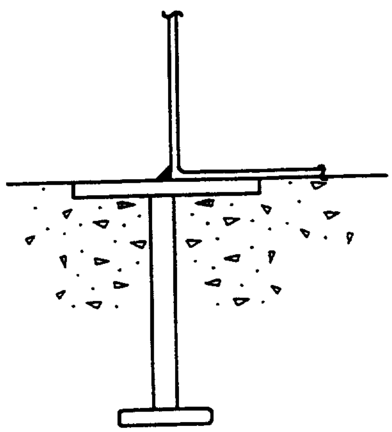
Figure 6.2-1 Equipment with Gap at Anchor Bolt (Figure 4-5 of SQUG GIP, Reference 1)



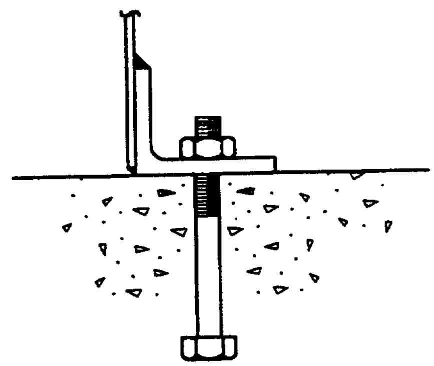
A Undesirable Flexible Welded Anchorage
Uplift causes sheet metal frame to bend.



B Undesirable Flexible Bolted Anchorage
Uplift causes sheet metal to bend.



C Desirable Stiff Welded Anchorage
Loads pass through sheet metal wall.



D Desirable Stiff Bolted Anchorage
Structural angle base provides adequate stiffness.

E Undesirable Flexible Base Anchorage
Plate can bend from uplift loads and base frame might bend from shear loads

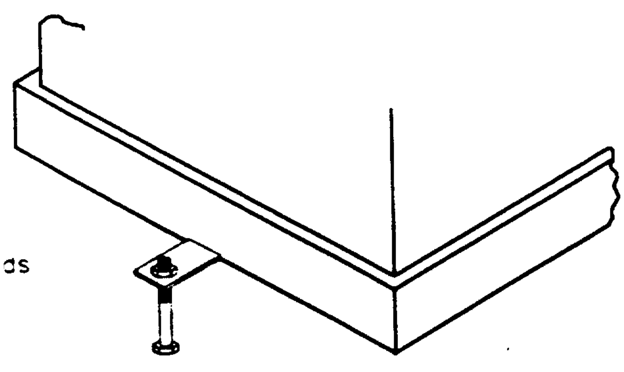


Figure 6.2-2 Examples of Stiff and Excessively Flexible Anchorage Connections (Reference 19) (Figure 4-6 of SQUG GIP, Reference 1)

6.3 ANCHORAGE CAPACITY DETERMINATION[8]

The next step in evaluating the seismic adequacy of anchorage is to determine the allowable capacity of the anchors used to secure an item of equipment. The allowable capacity is obtained by multiplying the nominal allowable capacities by the applicable capacity reduction factors. The nominal capacities and reduction factors can be obtained from this section.

The pullout capacity allowable is based on the product of the nominal pullout capacity and the applicable capacity reduction factors:

$$\boxed{P_{all} = P_{nom}\ RT_p\ RL_p\ RS_p\ RE_p\ RF_p\ RC_p\ RR_p\ RI_p}$$

Where:	P_{all}	=	Allowable Pullout capacity of installed anchor (kip)	
	P_{nom}	=	Nominal allowable Pullout capacity (kip)	
			expansion anchors	Section 6.3.1.1
			cast-in-place bolts and headed studs	Section 6.3.1.2
			cast-in-place J-bolts	Section 6.3.1.3
			grouted-in-place bolts	Section 6.3.1.4
			lead cinch anchors	Section 6.3.10.2
	RT_p	=	Reduction factor for the Type of expansion anchor	
			expansion anchors	Section 6.3.2
	RL_p	=	Reduction factor for short embedment Lengths	
			expansion anchors	Section 6.3.3.1
			cast-in-place bolts and headed studs	Section 6.3.3.2
			cast-in-place J-bolts	Section 6.3.3.3
			grouted-in-place bolts	Section 6.3.3.4
	RS_p	=	Reduction factor for closely Spaced anchors	
			expansion anchors	Section 6.3.4.1
			cast-in-place bolts and headed studs	Section 6.3.4.2
			cast-in-place J-bolts	Section 6.3.4.3
			grouted-in-place bolts	Section 6.3.4.4
	RE_p	=	Reduction factor for near Edge anchors	
			expansion anchors	Section 6.3.5.1
			cast-in-place bolts and headed studs	Section 6.3.5.2
			cast-in-place J-bolts	Section 6.3.5.3
			grouted-in-place bolts	Section 6.3.5.4

[8] Based on Section 4.4.2 of SQUG GIP (Ref. 1)

RF_p = Reduction factor for low strength concrete

expansion anchors	Section 6.3.6.1
cast-in-place bolts and headed studs	Section 6.3.6.2
cast-in-place J-bolts	Section 6.3.6.3
grouted-in-place bolts	Section 6.3.6.4

RC_p = Reduction factor for Cracked concrete

expansion anchors	Section 6.3.7.1
cast-in-place bolts and headed studs	Section 6.3.7.2
cast-in-place J-bolts	Section 6.3.7.3
grouted-in-place bolts	Section 6.3.7.4

RR_p = Reduction factor for expansion anchors securing equipment with essential Relays

expansion anchors	Section 6.3.8

RI_p = Reduction factor for reduced Inspection procedure

expansion anchors	Section 6.3.9.2

The shear capacity allowable is based on the product of the nominal shear capacity and the applicable capacity reduction factors:

$$\boxed{V_{all} = V_{nom}\, RT_s\, RL_s\, RS_s\, RE_s\, RF_s\, RR_s\, RI_s}$$

Where: V_{all} = Allowable shear capacity of installed anchor (kip)

V_{nom} = Nominal allowable shear capacity (kip)

expansion anchors	Section 6.3.1.1
cast-in-place bolts and headed studs	Section 6.3.1.2
cast-in-place J-bolts	Section 6.3.1.3
grouted-in-place bolts	Section 6.3.1.4
lead cinch anchors	Section 6.3.10.2

RT_s = Reduction factor for the Type of expansion anchor

expansion anchors	Section 6.3.2

RL_S = Reduction factor for short embedment Lengths

expansion anchors	Section 6.3.3.1
cast-in-place bolts and headed studs	Section 6.3.3.2
grouted-in-place bolts	Section 6.3.3.4

RS_s	=	Reduction factor for closely Spaced anchors	
		expansion anchors	Section 6.3.4.1
		cast-in-place bolts and headed studs	Section 6.3.4.2
		cast-in-place J-bolts	Section 6.3.4.3
		grouted-in-place bolts	Section 6.3.4.4
RE_s	=	Reduction factor for near Edge anchors	
		expansion anchors	Section 6.3.5.1
		cast-in-place bolts and headed studs	Section 6.3.5.2
		cast-in-place J-bolts	Section 6.3.5.3
		grouted-in-place bolts	Section 6.3.5.4
RF_s	=	Reduction factor for low strength concrete	
		expansion anchors	Section 6.3.6.1
		cast-in-place bolts and headed studs	Section 6.3.6.2
		cast-in-place J-bolts	Section 6.3.6.3
		grouted-in-place bolts	Section 6.3.6.4
RR_s	=	Reduction factor for expansion anchors securing equipment with essential Relays	
		expansion anchors	Section 6.3.8
RI_s	=	Reduction factor for reduced Inspection procedure	
		expansion anchors	Section 6.3.9.2

Note that the pullout and shear capacities for anchors given above are based on having adequate stiffness in the base of the equipment and on not applying significant prying action to the anchor. If Section 6.2 shows that stiffness is not adequate or that significant prying action is applied to the anchors, then the SCEs should lower the allowable capacity loads accordingly.

6.3.1 Type of Anchorage and Nominal Allowable Capacities[9]

It is important to identify which of these types of anchorage is used in an installation since these anchorage have different capacities and different installation parameters which should be checked during the inspection. The following four types of anchorage are covered in Sections 6.3.1 to 6.3.9:

1. Expansion Anchors - Shell and Nonshell Types
2. Cast-In-Place Bolts and Headed Studs
3. Cast-In-Place J-Bolts
4. Grouted-In-Place Bolts

Welds to embedded steel or exposed steel and lead cinch anchors are covered individually in Section 6.3.10. If any other type of anchorage is used to secure an item of equipment besides the four covered in this section and the other two covered in subsequent sections, the anchorage for that piece of equipment should be classified as an outlier and evaluated further in Chapter 12 or with the guidance in Reference 78.

In most cases, it will be necessary to use facility drawings, specifications, general notes, purchase records, manufacturer's data, or other such documents to identify the type of anchorage used for an item of equipment. Welds to embedded steel can be distinguished from bolted anchorage without using drawings; however, concrete drawings will still be needed to check the embedment details of the steel. It is not necessary to have specific documented evidence for each item of equipment installed in the facility; i.e., it is permissible to rely upon generic installation drawings or specifications so long as the SCEs have high confidence as to anchorage type and method of installation and remain alert for subtle differences in anchorage installations during the in-facility inspections. The SCEs should visually inspect the anchorage to check that the actual installation appears to be the same as that specified on the drawing or installation specification. If documents are not available to identify the type of bolted anchorage used for an installation, more detailed inspections should be done to develop a basis for the type of anchorage used and its adequacy.

For expansion anchors, it is important to identify the specific make and model of expansion anchor since there is considerable variance in seismic performance characteristics for different expansion anchor types. The makes and models of expansion anchors covered by this procedure are listed in Section 6.3.2 along with appropriate capacity reduction factors. Properly designed, deeply embedded cast-in-place headed studs and J-bolts have desirable performance since the failure mode is ductile, or steel governs. Well-designed and detailed welded connections to embedded plates or structural steel can provide a high-capacity anchorage. Special consideration should be given to grouted-in-place anchors since capacity is highly dependent on the installation practice used. If the grout shrinks any measurable amount, the anchor may have no tensile capacity.

[9] Based on Section 4.4.1 - Check 2 of SQUG GIP (Ref. 1)

6.3.1.1 Expansion Anchors[10]

The nominal allowable load capacities which can be used for the types of expansion anchors covered by this procedure (i.e., those listed in Section 6.3.2) are given in Table 6.3-1 below.

Table 6.3-1 Nominal Allowable Capacities for Expansion Anchors ($f'_c \geq 4000$ psi for Pullout and $f'_c \geq 3500$ psi for Shear)[1] (Table C.2-1 of SQUG GIP, Ref. 1)

Bolt/Stud Diameter (D, in.)	Pullout Capacity (P_{nom}, kip)	Shear Capacity (V_{nom}, kip)	Minimum Spacing[2] (S_{min}, in.)	Min. Edge Distance[2] (E_{min}, in.)
3/8	1.46	1.42	3.75	3.75
1/2	2.29	2.38	5.00	5.00
5/8	3.17	3.79	6.25	6.25
3/4	4.69	5.48	7.50	7.50
7/8	6.09	7.70	8.75	8.75
1	6.95	9.53	10.00	10.00

1 The pullout and shear capacities shown here are for the expansion anchor types included in Section 6.3.2 installed in sound, uncracked concrete (i.e., no cracks passing through the anchor bolt installation) with a compressive strength (f'_c) of at least 4000 psi for pullout and 3500 psi for shear.

2 Minimum spacings and edge distances are measured from bolt center to bolt center or concrete edge. Smaller spacings and edge distances less than the minimums given here can be used with the reduction factors given in Sections 6.3.4.1 and 6.3.5.1.

10 Based on Section C.2.1 of SQUG GIP (Ref. 1)

6.3.1.2 Cast-in-Place Bolts and Headed Studs[11]

The nominal allowable load capacities which can be used for cast-in-place bolts and headed studs are listed in Table 6.3-2.

Table 6.3-2 Nominal Allowable Capacities for Cast-In-Place Bolts and Headed Studs ($f'_c \geq 3500$psi)[1] (Table C.3-1 of SQUG GIP, Ref. 1)

Bolt/Stud Diameter (D, in.)	Pullout Capacity (P_{nom}, kip)	Shear Capacity (V_{nom}, kip)	Minimum Embedment[2] (L_{min}, in.)	Minimum Spacing[3] (S_{min}, in.)	Min. Edge Distance[3] (E_{min}, in.)
3/8	3.74	1.87	3-3/4	4-3/4	3-3/8
1/2	6.66	3.33	5	6-1/4	4-3/8
5/8	10.44	5.22	6-1/4	7-7/8	5-1/2
3/4	15.03	7.51	7-1/2	9-1/2	6-5/8
7/8	20.44	10.22	8-3/4	11	7-3/4
1	26.69	13.35	10	12-5/8	8-3/4
1-1/8	33.80	16.90	11-1/4	14-1/4	9-7/8
1-1/4	41.72	20.86	12-1/2	15-3/4	11
1-3/8	50.40	25.25	13-3/4	17-3/8	12-1/8

1 The pullout and shear capacities shown here are for ASTM A-307 (Ref. 79) or equivalent strength bolts installed in sound, uncracked concrete (i.e., no cracks passing through the anchor bolt installation) with a compressive strength of 3500 psi or greater. For bolt capacities in lower strength concrete see Section 6.3.6.2. For bolt capacities in cracked concrete see Section 6.3.7.2.

2 See Figure 6.3-1 for definition of embedment length (L). Smaller embedments than the minimum given here can be used with the reduction factor given in Section 6.3.3.2.

3 Minimum spacings and edge distances are measured from bolt center to bolt center or concrete edge. Spacings and edge distances less than the minimums given here can be used with the reduction factors given in Sections 6.3.4.2 and 6.3.5.2.

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11 Based on Section C.3.1 of SQUG GIP (Ref. 1)

6.3.1.3 Cast-in-Place J-Bolts[12]

The nominal allowable load capacities which can be used for cast-in-place J-bolts are listed in Table 6.3-3 below. The term J-bolt refers to a plain steel bar with a hook formed at the embedded end and threaded at the other end. An embedded bar can be considered as a J-bolt only if it has a hook on the embedded end meeting the minimum dimensions shown in Figure 6.3-2.

Table 6.3-3 Nominal Allowable Capacities for Cast-In-Place J-Bolts ($f'_c \geq 3500$ psi)[1] (Table C.4-1 of SQUG GIP, Ref. 1)

Bar Diameter (D, in.)	Pullout Capacity (P_{nom}, kip)	Shear Capacity (V_{nom}, kip)	Minimum Embedment[2] (L_{min}, in.) 180° Hook	90° Hook	Minimum Spacing[3] (S_{min}, in.)	Minimum Edge Distance[3] (E_{min}, in.)
3/8	3.74	1.87	16	20-1/2	1-1/8	3-3/8
1/2	6.66	3.33	21-1/4	27-1/4	1-1/2	4-3/8
5/8	10.44	5.22	26-5/8	34-1/8	1-7/8	5-1/2
3/4	15.03	7.51	31-7/8	40-7/8	2-1/4	6-5/8
7/8	20.44	10.22	37-1/4	47-3/4	2-5/8	7-3/4
1	26.69	13.35	42-1/2	54-1/2	3	8-3/4
1-1/8	33.80	16.90	47-7/8	61-3/8	3-3/8	9-7/8
1-1/4	41.72	20.86	53-1/8	68-1/8	3-3/4	11
1-3/8	50.40	25.25	58-1/2	75	4-1/8	12-1/8

1 The pullout and shear capacities shown here are from J-Bolts installed in sound, uncracked concrete with a compressive strength (f'_c) of at least 3500 psi.

2 Embedment length is defined in Figure 6.3-2.

3 Spacing and edge distance are measured from the center of the bolt(s).

[12] Based on Section C.4.1 of SQUG GIP (Ref. 1)

6.3.1.4 Grouted-in-Place Bolts[13]

The nominal allowable pullout and shear capacities which can be used for grouted-in-place bolts are listed in Table 6.3-4. Note that the values in this table are identical to those in Table 6.3-2 for cast-in-place bolts and headed studs except that the pullout capacities (P_{nom}) are reduced by a factor of 10. This was done since the pullout capacity of grouted-in-place bolts is significantly affected by the method of installation. Since documentation of the method used to install grouted-in-place bolts often is not available, the pullout capacities given in the table below are reduced significantly.

However, if the bolts were installed using effective installation procedures similar to those in Reference 80, then the pullout capacities of this grouted-in-place bolts may be taken to be the same as for cast-in-place bolts (i.e., use the capacities given in Table 6.3-2). Some of the installation techniques used in Reference 80 include such things as thorough cleansing of the concrete hole, acid etching of the concrete hole to roughen the surfaces, and use of grout which expands while it is curing.

Table 6.3-4 Nominal Allowable Capacities for Grouted-In-Place Bolts ($f'_c \geq 3500$ psi)[1] (Table C.5-1 of SQUG GIP, Ref. 1)

Bolt/Stud Diameter (D, in.)	Pullout Capacity[2] (P_{nom}, kip)	Shear Capacity (V_{nom}, kip)	Minimum Embedment[3] (L_{min}, in.)	Minimum Spacing[4] (S_{min}, in.)	Min. Edge Distance[4] (E_{min}, in.)
3/8	0.37	1.87	3-3/4	4-3/4	3-3/8
1/2	0.67	3.33	5	6-1/4	4-3/8
5/8	1.04	5.22	6-1/4	7-7/8	5-1/2
3/4	1.50	7.51	7-1/2	9-1/2	6-5/8
7/8	2.04	10.22	8-3/4	11	7-3/4
1	2.67	13.35	10	12-5/8	8-3/4
1-1/8	3.38	16.90	11-1/4	14-1/4	9-7/8
1-1/4	4.17	20.86	12-1/2	15-3/4	11
1-3/8	5.04	25.25	13-3/4	17-3/8	12-1/8

13 Based on Section C.5.1 of SQUG GIP (Ref. 1)

1 The pullout and shear capacities shown here are for ASTM A-307 (Ref. 79) or equivalent strength bolts installed in sound, uncracked concrete (i.e., no cracks passing through the anchor bolt installation) with a compressive strength of 3500 psi or greater. For bolt capacities in lower strength concrete see Section 6.3.6.3. For bolt capacities in cracked concrete see Section 6.3.7.3.

2 The pullout capacities (Pnom) are based on not having used special installation practices (or not knowing whether such practices were used). However, if installation procedures similar to those in Reference 80 were used, then the pullout capacities for cast-in-place bolts (Table 6.3-2) can be used in place of the values in this table.

3 See Figure 6.3-1 for definition of embedment length (L). Smaller embedments than the minimum given here can be used with the reduction factor given in Section 6.3.3.4.

4 Minimum spacings and edge distances are measured from bolt center to bolt center or concrete edge. Spacings and edge distances less than the minimums given here can be used with the reduction factors given in Sections 6.3.4.4 and 6.3.5.4.

6.3.2 Type of Expansion Anchor[14]

If the specific manufacturer and model of an expansion anchor is not known, then a generic capacity reduction factor as specified in Table 6.3-5 can be used. This generic factor may be used, however, only on expansion anchors made from carbon steel or better material. Concrete fasteners made from other materials or which use fastening mechanisms which are different than that of expansion anchors should be identified as outliers. This would include fasteners such as chemical anchors, plastic anchors, powder actuated fasteners, and concrete screws.

It is also important to distinguish between shell- and nonshell-type expansion anchors since different types of checks should be made to assure that they are properly installed. This section provides a description of the differences between shell and nonshell expansion anchors, how to tell them apart while they are installed, and what the capacity reduction factors are for the various makes and models. The shell type, or displacement controlled, (see Figure 6.3-3) and wedge type, or torque controlled, (see Figure 6.3-4) expansion anchors have been widely tested and have reasonably consistent capacity when properly installed in sound concrete.

Note that expansion anchors should generally not be used for securing vibratory equipment, such as pumps and air compressors. Expansion anchors used for vibrating equipment may rattle loose and have little to no tensile capacity. If such equipment is secured with expansion anchors, then there should be a large margin between the pullout loads and the pullout capacities; i.e., these expansion anchors should be loaded primarily in shear with very little pullout load. If a component which is secured with expansion anchors, has been in service for a long time and its expansion anchors remain tightly set, then this is a reasonable basis for ensuring installation adequacy. It is generally recommended that if expansion anchors need to be used for vibrating equipment, then the undercut-type of expansion anchors should be installed.

The specific manufacturers and product names of expansion anchors covered by this procedure are listed in Table 6.3-5 below. This table also lists capacity reduction factors (RT_p for pullout and RT_s for shear) which should be multiplied by the nominal pullout and shear capacities (P_{nom}, V_{nom}) given in Table 6.3-1.

RT_p = RT_s = Pullout (p) and shear (s) capacity reduction factors for type of expansion anchor from Table 6.3-5

[14] Based on Sections 4.1.1 - Check 2 and C.2.2 of SQUG GIP (Ref. 1) and information from Revision 3 of SQUG GIP (Ref. 12)

Table 6.3-5 Type of Expansion Anchors Covered by this Procedure and Associated Capacity Reduction Factors (Table C.2-2 of SQUG GIP, Ref. 1)

Manufacturer	Product Name	Type	Capacity Reduction Factors (RT_p,)	Capacity Reduction Factors (RT_S)
Drillco	MaxiBolt	Nonshell	1.0[1]	1.0[1]
Hilti	Kwik-Bolt	Nonshell	1.0	1.0[2]
	HDI	Shell	1.0	1.0[2]
	Sleeve (3/8 inch)	Nonshell	0.5[2]	1.0[2]
	Sleeve (1/2 to 5/8 inch)	Nonshell	0.75[2]	1.0[2]
ITW/Ramset	Dynaset	Shell	1.0	1.0[2]
	Dynabolt	Nonshell	0.75	0.75[2]
	Trubolt	Nonshell	0.75	0.75[2]
ITW/Ramset/ Redhead	Multiset Drop-In	Shell	1.0	1.0[2]
	Self Drilling	Shell	1.0	1.0[2]
	Dynabolt Sleeve	Nonshell	1.0	1.0[2]
	Nondrill	Shell	1.0	1.0[2]
	Stud	Shell	0.75	0.75[2]
	TRUBOLT	Nonshell	0.75	0.75[2]
Molly	Parasleeve	Nonshell	1.0	1.0[2]
	MDI	Shell	1.0	1.0[2]
	Parabolt	Nonshell	0.75	0.75[2]
Phillips	Self-Drilling	Shell	1.0	1.0[2]
	Wedge	Nonshell	1.0	1.0[2]
	Sleeve	Nonshell	1.0	1.0[2]
	Multi-Set	Shell	1.0	1.0[2]
	Stud	Shell	1.0	1.0[2]
	Non-Drilling	Shell	1.0	1.0[2]
Rawl	Drop-In	Shell	1.0	1.0[2]
	Stud	Shell	0.75	0.75[2]
	Saber-Tooth	Shell	0.75	0.75[2]
	Bolt	Nonshell	0.75	0.75[2]
Star	Selfdrill	Shell	0.75	0.75[2]
	Steel	Shell	0.75[2]	1.0[2]
	Stud	Shell	0.75[2]	0.75[2]
USE Diamond	Sup-R-Drop	Shell	1.0	1.0[2]
	Sup-R-Stud	Shell	1.0	1.0[2]
	Sup-R-Sleeve	Nonshell	1.0	1.0[2]
	Sup-R-Drill	Shell	0.75	0.75[2]
WEJ-IT	Drop-In	Shell	1.0	1.0[2]
	Sleeve	Nonshell	1.0	1.0[2]
	Wedge	Nonshell	0.5[2]	0.75[2]
	Stud	Shell	0.75[2]	1.0[2]
Unknown	Unknown (3/8 inch)[2]	Unknown[2]	0.5[2]	0.75[2]
	Unknown (> 3/8 inch)[2]	Unknown[2]	0.75[2]	0.75[2]

1 From Table C-2 of WSRC SEP-6 (Ref. 3)
2 From Table 6.3-5 of Revision 3 of SQUG GIP (Ref. 4), which is being reviewed by the NRC

If the specific manufacturer and product name of an expansion anchor is not known, then a generic capacity reduction factors as indicated below may be used:

$RT_P =$ 0.5 and $RT_S = 0.75$ (for bolt diameter = 3/8 inch)

$RT_P = 0.75$ and $RT_S = 0.75$ (for bolt diameter $\geq$ 3/8 inch)

Note, however, that this generic capacity reduction factor may only be used for expansion anchors made from carbon steel or better material. Concrete fasteners made from other materials or which use fastening mechanisms which are different than that of expansion anchors should be identified as outliers. This would include fasteners such as chemical anchors, plastic anchors, powder actuated fasteners, and concrete screws. "Unknown" anchors should be examined to ensure that they are not the WEJ-IT wedge anchor bolts, which can be distinguished from all other bolts by the two vertical slots cut along opposite sides of the bolt, parallel to the longitudinal axis of the bolt. Guidance on resolving anchorage outliers is provided in Reference 78.

In general, expansion anchors should not be used for securing vibratory equipment such as pumps and air compressors. If such equipment is secured with expansion anchors, then there should be a large margin between the pullout loads and the pullout capacities; i.e., the loads on these expansion anchors should be primarily shear.

The principal differences between shell- and nonshell-type expansion anchors are explained below.

Shell-type expansion anchors are expanded into the concrete by application of a setting force independent of the load later applied to the bolt or nut by the equipment being anchored. The key feature of this type of expansion anchor is that it relies upon its initial preset for holding it in place. Figure 6.3-3 shows the features of several types of shell-type expansion anchors.

Figure 6.3-3a shows a "Self-Drilling Type" of shell-type expansion anchor. This type of anchor is set in place by driving the shell down over the cone expander which is resting against the bottom of the hole.

Figure 6.3-3b shows a "Drop-In Type" which is set in place by driving a cone expander down through the center of the shell thereby causing the lower portion of the shell to expand into the concrete.

Figure 6.3-3c shows a "Phillips Stud Type" which is set in place by driving the stud down over the cone expander which is resting against the bottom of the hole.

Nonshell-type expansion anchors are expanded into the concrete by pulling the stud up out of the hole which causes a sleeve or a split ring to be forced into the concrete. The key feature of this type of expansion anchor is that the more the stud is loaded in tension, the greater the expansion setting force becomes. Figure 6.3-4 shows the features of two types of nonshell-type expansion anchors.

Figure 6.3-4a shows a "Sleeve Type" which is set in place by pulling the stud, with its integral cone expander on the bottom, up into the sleeve thereby forcing the lower split portion of the sleeve into the concrete. The sleeve is held in place during this setting process by butting up against the lower surface of the washer.

Figure 6.3-4b shows a "Wedge Type" which is set in place by pulling the stud, with its integral cone expander on the bottom, up though a split ring. Note that the split ring relies on friction against the concrete to stay in place during the setting operation.

Distinguishing characteristics of shell- and nonshell-type expansion anchors in their as-installed condition are shown in Figure 6.3-5.

Figure 6.3-5a shows a nonshell-type expansion anchor in which the visible portion is characterized by a smoothly cut or mechanically finished threaded stud with a nut holding the base of the equipment in place.

Figure 6.3-5b shows the most common type of shell-type expansion anchor in which the visible portion is characterized by a head of a bolt.

Figures 6.3-5c and 6.3-5d show other types of shell-type expansion anchors in which the visible portion is characterized by a rough cut or a raised knob on the end of the threaded rod. Careful inspection is necessary to distinguish these two types of shell expansion anchors from the nonshell-type shown in Figure 6.3-5a.

6.3.3 Embedment Length[15]

The embedment length of an anchor should be checked to confirm that it meets the minimum value so that nominal allowable anchor capacities can be used. A capacity reduction factor can be applied to the nominal allowable capacities for certain types of anchors with less embedment. Minimum embedments and reduction factors are given for each type of anchor covered in this procedure.

The minimum embedments for expansion anchors are based on the manufacturer's recommendations and cannot be reduced by applying capacity reduction factors. Expansion anchors which have deeper embedments may use the higher recommended capacities contained in the manufacturer's catalog in place of the nominal allowable capacities. The minimum embedments for cast-in-place bolts and headed studs and for grouted-in-place bolts are set to be sufficiently long so that the anchorage will fail in a ductile manner; i.e., in the bolt or stud, not in the concrete. Grouted-in-place anchor embedments are the same as those for cast-in-place anchors; a higher factor of safety is assigned to the pullout capacities of grouted-in-place anchors to account for uncertainties in the bolt installation. The minimum embedment for smooth bar J-bolts is based primarily on the bond strength between the bar and the concrete.

The embedment length of expansion anchors can be checked by confirming that the anchor is one of the makes and models covered by this procedure and performing a visual inspection of the installation. For many types of nonshell anchors, ultrasonic testing can be used to determine bolt length. Bolt embedment length may not be adequate if part of the shell is exposed or if there is a long stud protruding above the concrete surface.

It is not necessary to perform an embedment length check of an expansion anchor if the anchorage for that piece of equipment is robust, i.e., there is a large margin between the applied load and the anchorage capacity. Guidelines for evaluating whether there is sufficient margin in the anchorage are provided in Section 6.3.9.2, Reduced Inspection Alternative. The embedment length for anchor types other than expansion anchors can be determined from concrete installation drawings, ultrasonic testing, or other appropriate means.

6.3.3.1 Expansion Anchors[16]

If the embedment is greater than the values given in Table 6.3-6, then a pullout capacity reduction factor (RL_p) and a shear capacity reduction factor (RL_s) should be multiplied by the nominal pullout and shear capacities (P_{nom}, V_{nom}) given in Table 6.3-1.

$RL_p = RL_s$	=	Pullout (p) and shear (s) capacity reduction factors for expansion anchors	
	=	1.0	for embedments greater than those listed in Table 6.3-6
	=	Outlier	for embedments less than those listed in Table 6.3-6

(Note: This inspection check is not needed if the Reduced Inspection Alternative is chosen, as described in Section 6.3.9.2)

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15 Based on Section 4.4.1 - Check 5 of SQUG GIP (Ref. 1)

16 Based on Section C.2.4 of SQUG GIP (Ref. 1)

The manufacturer's recommended minimum embedments listed in Table 6.3-6 are from the catalogs of each of the vendors as listed in Reference 41, page E-27. These are the most recent catalogs available when Reference 41 was published. Expansion anchors with less than the minimum embedment should be documented as outliers. Guidance for resolving anchorage outliers is provided in Reference 78.

Table 6.3-6 Manufacturer's Recommended Minimum Embedment for Expansion Anchors Covered by this Procedure (Table C.2-6 of SQUG GIP, Ref. 1)

Manufacturer	Product Name (S=Shell, N=Nonshell)	Minimum Embedment (L) [in.] for Bolt/Stud Diameter: 3/8"	1/2"	5/8"	3/4"	7/8"	1"
Hilti	Kwik-Bolt (N)	1.63	2.25	2.75	3.25	--	4.50
	HDI (S)	1.56[1]	2.00	2.56[1]	3.19	--	--
	Sleeve (N)	1.50	2.00	2.00	--	--	--
ITW/Ramset	Dynaset (S)	1.63	2.00	2.63	3.25	--	--
	Dynabolt (N)	2.00	2.25	2.25	--	--	--
	Trubolt (N)	1.50	2.25	2.75	3.38	4.00	4.50
ITW/Ramset/	Multiset Drop-In (S)	1.63	2.00	2.50	3.19	--	--
Redhead	Self Drilling (S)	1.53	2.03	2.47	3.25	--	--
	Dynabolt Sleeve	1.88	2.00	2.25	--	--	--
	Nondrill (S)	1.56	2.06	2.56	3.19	--	--
	Stud (S)	1.63	1.88	2.38	2.88	--	--
	TRUBOLT (N)	1.50	2.25	2.75	3.25	3.75	4.50
Molly	Parasleeve (N)	1.50[1]	2.00[1]	2.00	--[1]	--	--
	MDI (S)	1.56[1]	2.00	2.50[1]	--[1]	--	--
	Parabolt (N)	1.50	2.25	2.75[1]	3.25	4.00	4.50

Table 6.3-6 (Continued)

Manufacturer	Product Name (S=Shell, N=Nonshell)	Minimum Embedment (L) [in.] for Bold/Stud Diameter: 3/8"	1/2"	5/8"	3/4"	7/8"	1"
Phillips	Self-Drilling (S)	1.53	2.03	2.47	3.25	3.69	--
	Wedge (N)	1.75	2.13	2.63	3.25	3.75	4.50
	Sleeve (N)	1.88	2.00	2.25	--	--	--
	Multi-Set (S)	1.38	1.75	2.25	2.50	--	--
	Stud (S)	1.63	1.88	2.38	2.88	--	--
	Non-Drilling (S)	1.56	2.06	2.56	3.19	--	--
Rawl	Drop-In (S)	1.88	2.38	3.00	3.50	--	--
	Stud (S)	1.75	2.25	2.88	3.38	4.00	4.50
	Saber Tooth (S)	1.53	2.03	2.47	3.25	3.69	--
	Bolt (N)	2.00	2.50	2.75	3.00	--	--
Star	Selfdrill (S)	1.53	2.03	2.47	3.25	3.69	--
	Steel (S)	1.44	1.94	2.38	3.00	--	--
	Stud (S)	1.63	1.75	2.38	2.88	--	--
USE Diamond	Sup-R-Drop (S)	1.56	2.00	2.53	3.19	--	--
	Sup-R-Stud (S)	2.16	2.81	3.31	4.25	4.72	5.56
	Sup-R-Sleeve (N)	1.50[1]	2.00[1]	2.50[1]	3.00[1]	--	--
	Sup-R-Drill (S)	1.53	2.03	2.47	3.27	--	--
WEJ-IT	Drop-In (S)	1.63	2.00	2.50	3.25	--	--
	Sleeve (N)	1.50	1.88	2.00	2.25	--	--
	Wedge (N)	1.50	2.00	3.00	3.00	4.50	5.50
	Stud (S)	1.75	2.13	3.63[1]	3.25	--	4.50

[1]From Table 6.3-6 of Revision 3 of SQUG GIP (Ref. 4), which is being reviewed by the NRC

These minimum embedments can be evaluated by performing the following inspection checks for shell- and nonshell-type expansion anchors. Note that these checks should be performed after the tightness check (Section 6.3.9) has been performed.

Shell-Type Expansion Anchors. The embedment length of shell-type expansion anchors is predetermined by the length of the shell and how it is installed in the concrete. The appropriate shell length is assured if the expansion anchor is one of the types listed in Table 6.3-6. An appropriate installation is assured if the shell of these anchors does not protrude above the surface of the concrete.

When making this embedment check, a check should also be made to confirm that the top of the shell is not touching the bottom of the base plate of the item of equipment being anchored. This check should be performed after the tightness check (Section 6.3.9) has been done. This will assure that the expansion anchor is tight in the hole and not just tight up against the base of the equipment.

If it is necessary to remove the bolt or nut from the anchorage to make the above two checks, then it is only necessary to spot check the embedment of a few anchors. If this spot check indicates that these types of bolts may not be properly installed, then this inspection check should be expanded accordingly. When re-installing the anchor, it should be re-tightened to a "wrench tight" condition or to the recommended tightness check torque values.

Nonshell-Type Expansion Anchors. The embedment length of nonshell-type expansion anchors is predetermined by the length of the stud and the installation of the anchor. The appropriate overall length of nonshell studs is dependent upon the manufacturer, the model, and the thickness of the equipment base plate for which the anchor is designed. Table 6.3-7, below, can be used as a generic screen for assessing whether a nonshell expansion anchor has adequate embedment. A range of projections is given in Table 6.3-7 since there are differences in acceptable projections depending upon the make and model of the anchor. If a nonshell stud projects more than the lower value of this range, then anchor-specific information should be used to determine the embedment length of the anchor.

Table 6.3-7 Maximum Stud Projections Above Concrete for Nonshell-Type Expansion Anchors (Table C.2-7 of SQUG GIP, Ref. 1)

Stud Diameter (in.)	Maximum Stud Projections Above Concrete (in.)
3/8	1/2 - 3/4
1/2	1/2 - 3/4
5/8	1/2 - 7/8
3/4	7/8 - 1 1/2
7/8	1 1/2 - 2
1	1 1/2 - 2

Note that careful evaluation is needed when checking the projections since larger projections than those given above may be needed if the base plate is relatively thick or if, at the time of installation in the facility, a particular bolt length may not have been available. Also, for bolts made by some manufacturers, the bolt projections may be larger than those given in the above table even for their shortest bolts. Thus, while this check need only be visual, a careful evaluation should be made to determine whether the stud projection is reasonable, given the bolt diameter, base plate thickness, and whether a grout pad is used. When projections are larger than those given in Table 6.3-7, adequate embedment should be evaluated by consulting design and construction documents and vendor catalogs. Alternately, ultrasonic inspection techniques may be used to compare the measured bolt/stud length to the manufacturer's recommended minimum embedment given in Table 6.3-6.

This embedment check should be performed on wedge- and sleeve-type, nonshell expansion anchors after the tightness check (Section 6.3.9) has been done. This is to ensure that the tightness check does not pull the expansion anchor partially out of the hole beyond the required minimum embedment.

For bolts with deeper embedments than the minimum values given in Table 6.3-6, manufacturer's catalog data may be used, if it is available, to establish the nominal allowable capacities instead of those given in Table 6.3-1. As an alternative, facility specific testing may be performed to establish the strength of the more deeply embedded expansion anchors. Guidance for resolving anchorage outliers is provided in Reference 78.

6.3.3.2 Cast-in-Place Bolts and Headed Studs[17]

The nominal pullout and shear capacities (P_{nom}, V_{nom}) given in Table 6.3-2 are based on the assumption that the embedment length is sufficiently long to preclude failure in the concrete. The minimum embedments (L_{min}) given in Table 6.3-2 are equal to 10 times the bolt diameter (D). Figure 6.3-1 shows the embedment length (L) for a cast-in-place bolt and a headed stud.

The embedment length should be evaluated by consulting existing drawings to ensure that the actual embedment length (L) is more than the minimum (L_{min}). If the construction drawings are not available, ultrasonic means or other appropriate methods may be used to evaluate the actual embedments.

If the embedment length (L) is less than the minimum value (L_{min}) given in Table 6.3-2, then a pullout capacity reduction factor (RL_p) and a shear capacity reduction factor (RL_s) should be multiplied by the nominal pullout and shear capacities (P_{nom}, V_{nom}) given in Table 6.3-2.

$RL_p = RL_s$ = Pullout (p) and shear (s) capacity reduction factors for cast-in-place anchors with shallow embedment

$$= 1.0 \qquad \text{for } L \geq 10D$$

$$= \frac{(L + D)\,L}{(L_{min} + D)\,L_{min}} \qquad \text{for } 4D < L < 10D \text{ and } L > 3 \text{ inches}$$

$$= \text{Outlier} \qquad \text{for } L < \text{Greater of: } 4D \text{ or } 3 \text{ inches}$$

17 Based on Section C.3.2 of SQUG GIP (Ref. 1)

L	=	Length of anchor embedment per Figure 6.3-1
L_{min}	=	Minimum length of anchor embedment from Table 6.3-2
D	=	Diameter of anchor bolt/stud

6.3.3.3 Cast-in-Place J-Bolts[18]

The nominal pullout capacities (P_{nom}) given in Table 6.3-3 are based on the assumption that the embedded length is at least as long as the minimum embedment lengths (L_{min}) given in Table 6.3.3.

If the embedment length (L) is less than the minimum value (L_{min}), then a pullout capacity reduction factor (RL_p) should be multiplied by the nominal pullout capacity (P_{nom}). A capacity reduction factor for shear is not needed since J-bolts develop their full shear strength even when the embedment is so small that the J-bolt becomes an outlier due to insufficient embedment for pullout (at L = 16D). Guidance for resolving anchorage outliers is provided in Reference 78.

RL_p	=	Pullout capacity reduction factor for cast-in-place J-bolts	
	=	1.0	for $L \geq L_{min}$
	=	$\dfrac{L + 20D}{62.5D}$	for 180° hook when $L_{min} > L \geq 16D$
	=	$\dfrac{L + 8D}{62.5D}$	for 90° hook when $L_{min} > L \geq 16D$
	=	Outlier	for $L < 16D$
L	=	Length of J-Bolt embedment per Figure 6.3-2 (in.)	
L_{min}	=	Minimum length of J-Bolt embedment from Table 6.3-3	
D	=	Rod diameter (in.)	

6.3.3.4 Grouted-in-Place Bolts[19]

For grouted-in-place bolts having embedments which are less than the minimum values given in Table 6.3-4, the capacity reduction factors given in Section 6.3.3.2 for cast-in-place bolts may be used to reduce the nominal pullout and shear capacities given in Table 6.3-4.

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18 Based on Section C.4.2 of SQUG GIP (Ref. 1)

19 Based on Section C.5.2 of SQUG GIP (Ref. 1)

6.3.4 Spacing Between Anchors[20]

The spacing from an anchor to each nearby anchor should be checked to confirm that it meets the minimum value so that nominal allowable anchor capacities can be used. A capacity reduction factor can be used when bolt-to-bolt spacing is less than the minimum specified value. Minimum spacings and reduction factors are given for each type of anchor covered in this procedure.

For expansion anchors, these spacing guidelines are based primarily on anchor capacity test results. The pullout capacity of cast-in-place anchors and headed studs is based on the shear cone theory. The minimum spacings are for distances between adjacent anchors in which the shear cones of the anchors overlap slightly, reducing the projected shear cone area for each anchor by about 13%. These minimum spacings are for anchors with the minimum embedment. Greater spacings are necessary to develop the full pullout capacities of deeply embedded anchors if higher capacity values are used. About 10 bolt diameter spacing is required to gain full capacity in expansion and cast-in-place anchors.

The shear capacity of anchors is not affected as significantly as tension capacity by closely-spaced anchors. Recommended minimum spacings for shear loads are given along with the corresponding capacity reduction factors for closely-spaced anchors.

For clusters of closely-spaced anchors, a capacity reduction factor should be applied to an anchor for every other nearby anchor. For example, if there are three anchors in a line and all are closer than the minimum spacing, then the center anchor should have two reduction factors applied to its nominal capacity allowable and the outside anchors should have only one reduction factor applied.

The spacings between anchors can be checked in the field by a visual inspection and, if necessary, the spacings can be measured. Measurements should be made from anchor centerline to anchor centerline.

6.3.4.1 Expansion Anchors[21]

If the spacing (S) between an expansion anchor and another anchor is less than the minimum value (S_{min}) given in Table 6.3-1, then a pullout capacity reduction factor (RS_p) and a shear capacity reduction factor (RS_s) should be multiplied by the nominal pullout and shear capacities (P_{nom}, V_{nom}) given in Table 6.3-1.

RS_p = Pullout capacity reduction factor for closely spaced expansion anchors

$= 1.0 \quad \text{for } S \geq 10D$

$= \dfrac{S}{10D} \quad \text{for } 10D > S \geq 5D$

$= 0.5 \quad \text{for } 5D > S \geq 2.5D$

= Outlier for $S < 2.5D$

S = Spacing between anchors measured center-to-center

20 Based on Section 4.4.1 - Check 7 of SQUG GIP (Ref. 1)

21 Based on Section C.2.5 of SQUG GIP (Ref. 1)

D	=	Diameter of anchor bolt/stud
RS_s	=	Shear capacity reduction factor for closely spaced expansion anchors
	=	1.0 for $S \geq 2D$
	=	0.5 for $S < 2D$

A reduction factor should be applied for each nearby anchor, whether it is another expansion anchor or a different type of anchor. The spacings (S) given above are defined in terms of multiples of the anchor bolt/stud diameter (D), measured from anchor centerline to centerline.

6.3.4.2 Cast-in-Place Bolts and Headed Studs[22]

If the spacing (S) between a cast-in-place anchor and another anchor is less than the minimum value (S_{min}) given in Table 6.3-2, then a pullout capacity reduction factor (RS_p) and a shear capacity reduction factor (RS_s) should be multiplied by the nominal pullout and shear capacities (P_{nom}, V_{nom}) given in Table 6.3-2.

Note that a reduction factor should be applied for each nearby anchor, whether it is another cast-in-place anchor or a different type of anchor. For example, for 4 bolts in a line, the interior bolts would be subject to 2 reductions, while the exterior bolts would be subject to only one reduction.

Note that if there are 5 or more cast-in-place anchors in a cluster which are spaced closer together than the minimum (S_{min}) as defined in Table 6.3-2, then the pullout capacity reduction factor (RS_p) cannot be used and the anchors in that cluster should instead be identified as outliers.

RS_p	=	Pullout capacity reduction factor for closely spaced cast-in-place anchors	
	=	1.0	for $S \geq S_{min}$
	=	$\dfrac{A_{s,red}}{A_{s,nom}}$	for $S < S_{min}$
	=	Outlier	where there are 5 or more cast-in-place anchors in a cluster in which $S < S_{min}$
S	=	Spacing from the bolt being evaluated to an adjacent bolt measured center-to-center	
S_{min}	=	Minimum spacing to develop full pullout strength from Table 6.3-2	

22 Based on Section C.3.3 of SQUG GIP (Ref. 1)

$A_{s,nom}$ = Nominal projected area of the nonoverlapping shear cone of a single bolt located at the minimum spacing distance (S_{min}) from Table 6.3-8. The values of $A_{s,nom}$ given in Table 6.3-8 are about 13 percent less than the full, geometric shear cone projected area.

Table 6.3-8 Nonoverlapping Projected Shear Cone Areas for Bolts Meeting Minimum Spacing Requirements (Table C.3-2 of SQUG GIP, Ref. 1)

Bolt Diameter (D, In.)	Nonoverlapping Shear Cone Area ($A_{s,nom}$, in.^{2})
3/8	41.9
1/2	74.1
5/8	116.0
3/4	167.4
7/8	227.2
1	297.3
1-1/8	376.7
1-1/4	464.1
1-3/8	562.2

$A_{s,red}$ = Reduced projected area of the nonoverlapping shear cone of a single bolt located less than the minimum spacing (S_{min}) from another bolt. The values of $A_{s,red}$ are calculated from the following equation:

$$= \pi r^2 - \frac{1}{2}\left[r^2\ \theta - r\ S\ \sin\left(\frac{\theta}{2}\right)\right]$$

$$r = \frac{2L + D}{2}$$

$$\theta = 2\cos^{-1}\left[\frac{S}{2L + D}\right]$$

S	=	Spacing between bolt being evaluated and adjacent bolt measured center-to-center	
L	=	Length of embedment of bolt being evaluated	
D	=	Diameter of anchor bolt/stud	
RS_s	=	Shear capacity reduction factor for closely spaced cast-in-place anchors.	
	=	1.0	for $S \geq 2D$
	=	0.5	for $S < 2D$

6.3.4.4 Cast-in-Place J-Bolts[23]

The nominal shear capacities (V_{nom}) for J-bolts given in Table 6.3-3 are based on a minimum spacing of 3D, where D is the diameter of the J-bolt.

For spacings less than 3D, the J-bolt is an outlier.

6.3.4.4 Grouted-in-Place Bolts[24]

For grouted-in-place bolts having, spacings which are less than the minimum values given in Table 6.3-4, the capacity reduction factors given in Section 6.3.4.2 for cast-in-place bolts may be used to reduce the nominal pullout and shear capacities given in Table 6.3-4.

23 Based on Section C.4.3 of SQUG GIP (Ref. 1)
24 Based on Section C.5.2 of SQUG GIP (Ref. 1)

6.3.5 Edge Distance[25]

The distance from an anchor to a free edge of concrete should be checked to confirm that it meets the minimum value so that the nominal allowable anchor capacities can be used. A capacity reduction factor can be used for an anchor which is closer to an edge than the minimum. Minimum edge distances and reduction factors are given for each type of anchor covered in this procedure.

For expansion anchors, these edge distance guidelines are based primarily on anchor capacity test results. Full pullout and shear capacity can be developed for cast-in-place anchors and headed studs which are no closer to a free edge than the radius of the projected shear cone. The minimum edge distances correspond to the shear cone just touching the free edge of concrete at the surface (no credit is taken for concrete reinforcement). These minimum edge distances apply to anchors with the minimum embedment. Greater edge distances are necessary to develop the full pullout capacities of deeply embedded anchors if higher capacities are used. About 10 bolt diameter edge distance is required to gain full capacity of expansion anchors.

When an anchor is near more than one free concrete edge, a capacity reduction factor should be applied for each nearby edge. For example, if an anchor is near a corner, then two reduction factors apply. The edge distances can be checked in the field by a visual inspection and, if necessary, the edge distances can be measured. Measurements should be made from anchor centerline to the free edge.

6.3.5.1 Expansion Anchors[26]

If the distance (E) from an expansion anchor to a free edge of concrete is less than the minimum value (E_{min}) given in Table 6.3-1, then a pullout capacity reduction factor (RE_p) and a shear capacity reduction factor (RE_s) should be multiplied by the nominal pullout and shear capacities (P_{nom}, V_{nom}) given in Table 6.3-1.

RE_p = Pullout capacity reduction factor for near edge expansion anchors

$$= 1.0 \quad \text{for} \quad E \geq 10D$$

$$= \frac{E}{10D} \quad \text{for} \quad 10D > E \geq 4D$$

$$= 0.0 \text{ (Outlier)} \quad \text{for} \quad E < 4D$$

E = Edge distance from centerline of anchor to free edge

D = Diameter of anchor bolt/stud

25 Based on Section 4.4.1 - Check 8 of SQUG GIP (Ref. 1)

26 Based on Section C.2.6 of SQUG GIP (Ref. 1)

RE_s	=	Shear capacity reduction factor for near edge expansion anchors	
	=	1.0	for $E \geq 10D$
	=	$\left[\dfrac{E}{10D}\right]^{1.5}$	for $10D > E \geq 4D$
	=	0.0 (Outlier)	for $E < 4D$

A reduction factor should be applied for each nearby edge; e.g., if an anchor is near a corner, then two reduction factors apply. The edge distance (E) given in the tables above are in terms of multiples of the anchor bolt/stud diameter (D), measured from the anchor centerline to the edge.

6.3.5.2 Cast-in-Place Bolts and Headed Studs[27]

If the distance (E) from a cast-in-place bolt or a headed stud to a free edge of concrete is less than the minimum value (E_{min}), given in Table 6.3-2, then a pullout capacity reduction factor (RE_p) and a shear capacity reduction factor (RE_s) should be multiplied by the nominal pullout and shear capacities (P_{nom}, V_{nom}), given in Table 6.3-2. A reduction factor should be applied for each nearby edge; e.g., if an anchor is near a corner, then two reduction factors apply.

RE_p	=	Pullout capacity reduction factor for near edge cast-in-place bolts and headed studs	
	=	1.0	for $E \geq E_{min}$
	=	$\dfrac{A_{e,red}}{A_{e,nom}}$	for $E_{min} > E \geq 4D$
	=	0.0 (Outlier)	for $E < 4D$
E	=	Edge distance from centerline of anchor to free edge	
E_{min}	=	Minimum edge distance to develop full pullout capacity from Table 6.3-2	
D	=	Diameter of anchor bolt/stud	
$A_{e,nom}$	=	Nominal projected shear cone area of a bolt which is located away from a free concrete edge at least the minimum edge distance (E_{min}) given in Table 6.3-2	
	=	$0.96 \dfrac{\pi}{4} (2L + D)^2$	
L	=	Length of embedment of bolt being evaluated	

27 Based on Section C.3.4 of SQUG GIP (Ref. 1)

$A_{e,red}$ = Reduced projected shear cone area of a bolt located at less than the minimum edge distance from a concrete edge

$$= \pi r^2 - \frac{1}{2}\left[r^2\theta - 2r\ E\ \sin\left(\frac{\theta}{2}\right)\right]$$

$$\theta = 2\cos^{-1}\left[\frac{2E}{2L + D}\right]$$

$$r = \frac{2L + D}{2}$$

RE_s = Shear capacity reduction factor for near edge cast-in-place bolts and headed studs

$$= 1.0 \qquad \text{for} \quad E \geq 8.75D$$

$$= 0.0131\left[\frac{E}{D}\right]^2 \qquad \text{for} \quad 8.75D > E \geq 4D$$

$$= 0.0\ \text{(Outlier)} \qquad \text{for} \quad E < 4D$$

6.3.5.3 Cast-in-Place J-Bolts[28]

The minimum edge distances given in Table 6.3-3 for J-bolts are the same as those for cast-in-place bolts and headed studs. Likewise the capacity reduction factors for J-bolts installed near an edge are also the same as discussed in Section 6.3.5.2 for cast-in-place bolts and headed studs.

For calculating reduction factors for near-edge J-bolts, the "L" dimension from Table 6.3-2 for cast-in-place bolts should be used.

6.3.5.4 Grouted-in-Place Bolts[29]

For grouted-in-place bolts having edge distances which are less than the minimum values given in Table 6.3-4, the capacity reduction factors given in Section 6.3.5.2 for cast-in-place bolts may be used to reduce the nominal pullout and shear capacities given in Table 6.3-4.

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28 Based on Section C.4.4 of SQUG GIP (Ref. 1)
29 Based on Section C.5.2 of SQUG GIP (Ref. 1)

6.3.6 Concrete Strength and Condition[30]

The concrete compressive strength (f_c') should be obtained from design documentation or tests to confirm that it meets the minimum value so that the nominal allowable anchor capacities can be used. A capacity reduction factor can be used for concrete which has lower strength than the minimum. Minimum concrete strength and reduction factors are given for each type of anchor covered in this procedure.

In addition, the concrete in the vicinity of the anchor should be checked to be sure that it is free of gross defects which could affect the holding strength of the anchor. This check should be done in conjunction with Section 6.3.7. Surface defects such as hairline shrinkage cracks are not of concern.

Note that this procedure covers anchors installed only in poured, structural concrete. If any equipment is secured to other types of concrete or masonry structures, such as concrete block masonry walls, the anchorage for that item of equipment should be classified as an outlier and evaluated separately using guidance in Chapter 12 and Reference 78.

The compressive strength of the concrete can normally be obtained from facility construction drawings, specifications, or other documents. If this information is not available, core sample information can be used or new samples can be taken and tested.

Expansion anchors installed in masonry block walls have lower capacity than those in concrete and should be classified as outliers. Block wall adequacy (anchorage and reinforcement) should be checked as part of the outlier resolution.

6.3.6.1 Expansion Anchors[31]

If the concrete compressive strength (f_c') is less than 4000 psi for pullout loads or 3500 psi for shear loads, then a pullout capacity reduction factor (RF_p) and a shear capacity reduction factor (RF_s) should be multiplied by the nominal pullout and shear capacities (P_{nom}, V_{nom}), given in Table 6.3-1.

RF_p = Pullout capacity reduction factor for expansion anchors in low strength concrete

$$RF_p = \begin{cases} 1.0 & \text{for } f_c' \geq 4000 \text{ psi} \\ \dfrac{f_c'}{4000} & \text{for } 4000 \text{ psi} > f_c' \geq 2000 \text{ psi} \\ \text{Outlier} & \text{for } f_c' < 2000 \text{ psi} \end{cases}$$

f_c' = Concrete compression strength (psi)

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30 Based on Section 4.4.1 - Check 9 of SQUG GIP (Ref. 1)

31 Based on Section C.2.7 of SQUG GIP (Ref. 1)

RF_s = Shear capacity reduction factor for expansion anchors in low strength concrete

$= 1.0$ for $f_c' \geq 3500$ psi

$= \dfrac{f_c'}{10{,}000} + 0.65$ for $3500 \text{ psi} > f_c' \geq 2000$ psi

= Outlier for $f_c' < 2000$ psi

6.3.6.2 Cast-in-Place Bolts and Headed Studs[32]

If the concrete compressive strength (f_c') is less than 3500 psi, then a pullout capacity reduction factor (RF_p) and a shear capacity reduction factor (RF_s) should be multiplied by the nominal pullout and shear capacities (P_{nom}, V_{nom}) given in Table 6.3-2.

$RF_p = RF_s$ = Pullout (p) and shear (s) capacity reduction factors for cast-in-place bolts and headed studs in low strength concrete

$= 1.0$ for $f_c' \leq 3500$ psi

$= \sqrt{\dfrac{f_c'}{3500}}$ for $3500 \text{ psi} > f_c' \geq 2500 \text{ psi}$

= Outlier for $f_c' < 2500$ psi

f_c' = Concrete compressive strength (psi)

32 Based on Section C.3.5 of SQUG GIP (Ref. 1)

6.3.6.3 Cast-in-Place J-Bolts[33]

If the concrete compressive strength (f_c') is less than 3500 psi, then a pullout capacity reduction factor (RF_p) and a shear capacity reduction factor (RF_s) should be multiplied by the nominal pullout and shear capacities (P_{nom}, V_{nom}) given in Table 6.3-3.

$RF_p = RF_s$ = Pullout (p) and shear (s) capacity reduction factors for J-bolts in low strength concrete

$$= 1.0 \quad \text{for} \quad f_c' \geq 3500 \text{ psi}$$

$$= \sqrt{\frac{f_c'}{3500}} \quad \text{for} \quad 2500 \text{ psi} \leq f_c' < 3500 \text{ psi}$$

$$= \text{Outlier} \quad \text{for} \quad f_c' < 2500 \text{ psi}$$

f_c' = Concrete compressive strength (psi)

6.3.6.4 Grouted-in-Place Bolts[34]

When grouted-in-place bolts are installed in concrete which has a compressive strength of $f_c' \leq 3500$ psi, the capacity reduction factors given in Section 6.3.6.2 for cast-in-place bolts may be used to reduce the nominal pullout and shear capacities given in Table 6.3-4.

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33 Based on Section C.4.5 of SQUG GIP (Ref. 1)
34 Based on Section C.5.3 of SQUG GIP (Ref. 1)

6.3.7 Concrete Crack Locations and Sizes[35]

The concrete should be checked to confirm that it is free of significant structural cracks in the vicinity of the installed anchors so that the nominal pullout capacities can be used. A pullout capacity reduction factor can be used for concrete which has cracks which are larger than the acceptable maximum widths and are located in the vicinity of the anchor. Maximum acceptable crack sizes and capacity reduction factors are given for each type of anchor covered in this procedure.

Significant structural cracks in concrete are those which appear at the concrete surface and pass through the concrete shear cone of an anchor installation or the location of the expansion wedge. Concrete with surface (craze) cracks or shrinkage cracks which only affect the surface of the concrete should be considered uncracked.

The check for cracks in the concrete can be done by a visual inspection of the anchorage installation. It may be necessary to exercise judgment to establish whether cracks in the vicinity of an anchor actually pass through the installation. It is sufficient to estimate the width of cracks without making detailed measurements. This check should be done in conjunction with Section 6.3.6 to find other gross defects which could affect the holding strength of an anchor.

6.3.7.1 Expansion Anchors[36]

If there are significant structural cracks in the concrete where expansion anchors are installed, then a pullout capacity reduction factor (RC_p) should be multiplied by the nominal pullout capacity (P_{nom}), given in Table 6.3-1. The shear capacity of expansion anchors is not significantly affected by cracks in the concrete.

RC_p = Pullout capacity reduction factor for expansion anchors in cracked concrete

= See Table 6.3-9 for values

The pullout capacity reduction factor applies only to significant structural cracks which penetrate the concrete mass and pass through the vicinity of the anchor installation. Concrete with surface (craze) cracks or shrinkage cracks which only affect the surface of the concrete should be considered uncracked. It may be necessary to exercise judgment to establish whether cracks in the vicinity of an anchor actually pass through the installation. Inspections for crack width should be visual (i.e., detailed measurement of crack widths is not necessary).

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[35] Based on Section 4.4.1 - Check 10 of SQUG GIP (Ref. 1)

[36] Based on Section C.2.8 of SQUG GIP (Ref. 1)

Table 6.3-9 Pullout Capacity Reduction Factors for Expansion Anchors in Cracked Concrete (Table C.2-8 of SQUG GIP, Ref. 1)

Conditions	Reduction Factor for Pullout Capacity (RC_p)
No Cracks	1.0
Crack Size < 0.01 in. and the number of required anchors securing the equipment which are affected by these cracks is:	
≤ 50%	1.0
> 50%	0.75*
0.01 in. ≤ Crack Size ≤ 0.02 in	0.75*
Crack Size > 0.02 in.	Outlier

* Capacity reduction factor applies to all required anchors securing the item of equipment, not just the anchors which are affected by the cracks.

6.3.7.2 Cast-in-Place Bolts and Headed Studs[37]

If there are significant structural cracks in the concrete where the cast-in-place bolts and headed studs are installed, then a pullout capacity reduction factor (RC_p) should be multiplied by the nominal pullout capacity (P_{nom}) given in Table 6.3-2. The shear capacity of the cast-in-place bolts and headed stud anchors is not significantly affected by cracks in the concrete.

The pullout capacity reduction factor applies only to significant structural cracks which penetrate the concrete mass and pass through the vicinity of the anchor installation. Concrete with surface (craze) cracks or shrinkage cracks which only affect the surface of the concrete should be considered uncracked. It may be necessary to exercise judgment to establish whether cracks in the vicinity of an anchor actually pass through the installation. Inspections for crack width should be visual (i.e., detailed measurement of crack widths is not necessary).

RC_p	=	Pullout capacity reduction factor for cast-in-place anchors in cracked concrete	
	=	1.0	for no cracks and for CS < 0.01 in.
	=	1.08 - 8 CS	for 0.01 in. ≤ CS ≤ 0.06 in.
	=	Outlier	for CS > 0.06 in.
CS	=	Crack size (approximate size based on visual observation)	

[37] Based on Section C.3.6 of SQUG GIP (Ref. 1)

6.3.7.3 Cast-in-Place J-Bolts[38]

The areas adjacent to J-bolt installations should be inspected for significant structural cracks which penetrate the concrete mass. Concrete with surface (craze) cracks or shrinkage cracks which only affect the surface of the concrete should be considered uncracked. Inspections for crack width should be visual (i.e., detailed measurement of crack widths is not necessary). J-bolts should be classified as outliers when either of the following two crack sizes are exceeded:

- When cracks are larger than about 0.02 inch wide and traverse through the J-bolt installation, or

- When cracks are larger than about 0.05 inches wide and exist near the J-bolt installation.

6.3.7.4 Grouted-in-Place Bolts[39]

If there are significant structural cracks in the concrete where the grouted-in-place bolts are installed, then the pullout capacity reduction factors given in Section 6.3.7.2 for cast-in-place bolts may be used to reduce the nominal pullout capacities given in Table 6.3-4.

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38 Based on Section C.4.6 of SQUG GIP (Ref. 1)

39 Based on Section C.5.3 of SQUG GIP (Ref. 1)

6.3.8 Essential Relays in Cabinets[40]

Electrical cabinets and other equipment which are secured with expansion anchors should be checked to determine whether they house essential relays. If essential relays are present, a capacity reduction factor of 0.75 should be used for cabinets which are secured with expansion anchors. The check for the presence of essential relays in equipment can be done in conjunction with the Relay Functionality Review described in Chapter 11.

The basis for this capacity reduction factor is that expansion anchors have a tendency to loosen slightly when they are heavily loaded (i.e., they pull out of the concrete slightly). This effect does not significantly reduce the ultimate load carrying capability of expansion anchors; however, the slight gap between the base of the equipment and the surface of the concrete can open during the first part of an earthquake load cycle and then slam closed during the second part of the cycle. This creates high frequency impact loadings on the equipment, and the relays mounted therein could chatter. Use of a capacity reduction factor for the expansion anchors which secure this type of equipment lowers the maximum load which the anchor will experience; therefore this minimizes the amount of loosening and hence the potential for introducing high frequency impact loadings into the equipment.

If there are essential relays mounted in the item of equipment, then the following pullout capacity reduction factor (RR_p) and shear capacity reduction factor (RR_s) should be multiplied by the nominal pullout and shear capacities (P_{nom}, V_{nom}) given in Table 6.3-1.

RR_p = Pullout capacity reduction factor for expansion anchors securing equipment in which essential relays are mounted

= 0.75

RR_s = Shear capacity reduction factor for expansion anchors securing equipment in which essential relays are mounted

= 0.75

The Relay Functionality Review described in Chapter 11 identifies which cabinets and items of equipment contain essential relays.

40 Based on Sections 4.4.1 - Check 11 and C.2.9 of SQUG GIP (Ref. 1)

6.3.9 Tightness Check and Reduced Inspection Procedure for Expansion Anchors

6.3.9.1 Tightness Check for Expansion Anchors[41]

The tightness check for expansion anchors can be accomplished by applying a torque to the anchor by hand until the anchor is "wrench tight," i.e., tightened without excessive exertion. If the anchor bolt or nut rotates less than about 1/4 turn, then the anchor is considered tight. This tightness check is not intended to be a proof test of the capacity of the anchorage. This check is merely meant to provide a reasonable assurance that the expansion anchor is not loose in the hole due to gross installation defects. Loose nuts may indicate inadequate anchor set.

It is not the intent of this procedure to require disassembly of cabinets and structures or removal of electrical cabling and conduit to provide access to the expansion anchors for this tightness check. Therefore, in those cases where expansion anchors are inaccessible, either during facility operation or during shutdown, the SCEs should make a judgment as to whether the number and distribution of tightness checks which have already been made in the facility is sufficient, considering both the problem of inaccessibility and the results of the other tightness checks. One concern with not checking the tightness of inaccessible expansion anchors is that these types of anchors may not have been properly installed because access to them was limited during installation; therefore, the reason for inaccessibility should be considered when deciding not to check the tightness of expansion anchors.

For facilities which have a large number of similar expansion anchors installed, a sampling program may be used for the tightness check based on achieving 95% confidence that no more than 5% of the expansion anchors fail the tightness test. Guidelines for conducting a sampling program are provided below.

It is not necessary to perform a tightness check of an expansion anchor if the anchorage for that piece of equipment is robust; i.e., there is a large margin between the applied load and the anchorage capacity. Guidelines for evaluating whether there is sufficient margin in the anchorage are provided below.

It is not necessary to perform a tightness check of expansion anchors which are loaded in tension due to dead weight, since the adequacy of the anchor set is effectively proof-tested by the dead weight loading. Judgment should be exercised to assess the need for tightness checks when multiple expansion anchors are used to secure a base plate loaded in tension by dead loads.

SCEs should be aware that a tightness check alone for shell-type expansion anchors may not be sufficient to detect gross installation defects of expansion anchors. If the top of the shell is in contact with the equipment base, then the tightness check may simply be tightening the shell against the bottom of the equipment base as shown in Figure 6.3-6. SCEs should exercise engineering judgment and spot check for this type of installation defect by removing a few bolts from shell-type anchors and inspecting them to ensure that the shell anchor and the equipment base are not in contact. If this spot check indicates that these types of bolts may not be properly installed, then the inspection check should be expanded accordingly. Embedment length is determined from the point on the anchor to the surface of the structural concrete. Grout pads should not be included in the embedment length.

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41 Based on Sections 4.4.1 - Check 4 and C.2.3 of SQUG GIP (Ref. 1)

The tightness check can be performed by using a standard size box or open-end wrench on the bolt head or nut and applying a torque by hand until the bolt or nut is "wrench tight"; i.e., tightened without excessive exertion. For those cases where specific torque values must be used (e.g., for maintenance work orders), the "Tightness Check Torque" values given in Table 6.3-10, below, can be used for this expansion anchor tightness check. These values correspond to about 20% of the normal installation torques.

Table 6.3-10 Recommended Torque Values for Expansion Anchor Tightness Check (Table C.2-3 of SQUG GIP, Ref. 1)

Anchor Diameter (in.)	Installation Torque (ft-lbs)	Tightness Check Torque (ft-lbs)
3/8	25-35	5-7
1/2	45-65	9-13
5/8	80-90	16-18
3/4	125-175	25-35
7/8	200-250	40-50
1	250-300	50-60
1-1/4[1]	400-500[1]	80-100[1]

[1]Data from Table C-39 of WSRC SEP-6 (Ref. 3)

A well-installed expansion anchor should not rotate under this applied torque. A small amount of initial rotation (about 1/4 turn) is acceptable provided the nut or bolt will tighten and resist the applied torque. If a bolt turns more than about 1/4 turn, but does eventually resist the torque, it should be re-torqued to the manufacturer's recommended installation torque and then considered acceptable.

A sampling program can be used to check the tightness of expansion anchors provided it achieves 95% confidence that no more than 5% of the expansion anchors fail to meet the tightness guidelines given above. This 95/5 criterion can be met using the guidelines given below for sample size, homogeneous population, allowable number of nonconforming anchors, and use of initial tightness test results.

- Sample Size. The number of expansion anchors selected for tightness checking should be at least as large as given in Table 6.3-11 below for "Sample Size".

Table 6.3-11 Sample Size for Expansion Anchor Tightness Check (Table C.2-4 of SQUG GIP, Ref. 1)

Condition	Sample[1] Size
Expansion Anchors Securing Equipment Which Contains Essential Relays	100%
Total Size of Homogeneous Anchor Population Is Less Than 40 Anchors	100%
Total Size of Homogeneous Anchor Population Is Between 40 and 160 Anchors	40 Anchors
Total Size of Homogeneous Anchor Population Is More Than 160 Anchors	20%

[1]Note: The sample sizes provided in this table are for accessible bolts.

- Homogeneous Population. The sample size is based on the total population of expansion anchors being homogeneous. Factors such as installation specifications, quality assurance procedures used in the installation specifications, quality assurance procedures used during installation, bolt manufacturer, installation contractor, etc., should be considered when judging whether or not the total population is homogeneous. If there is more than one homogeneous set of expansion anchors, then the sample size limitations given above and the allowable number of nonconforming anchors given below apply to each individual population.

- Allowable Number of Nonconforming Anchors. The criterion of 95% confidence that there are no more than 5% nonconforming anchors can be met if the number of expansion anchors which fails the tightness check does not exceed the limitations given in Table 6.3-12 below. If more than these number of anchors fail the tightness check, then the sample size should be increased until the failure rate does not exceed the limitations in this table.

- Use of Initial Tightness Test Results. The results of the initial torque tightness check on each expansion anchor should be used to establish the failure rate for the purposes of the sampling program. For example, if out of a total population of 400 expansion anchors 100 were tightness checked and 4 of these failed the initial check, then the sample size should be expanded. (Table 6.3-12 only allows 3 anchors to fail for 100 tests on a population of 400.) The sample size should be expanded even if all 4 of the failed anchors were able to be fully tightened up to their installation torque requirements.

Table 6.3-12 Allowable Number of Expansion Anchors Which Need Not Pass Tightness Check (Table C.2-5 of SQUG GIP, Ref. 1)

Total Population	Number of Anchors Which Need Not Pass Tightness Check for Test Sample Size, (n):											
Size N	40	60	80	100	150	200	250	300	350	400	450	500
100	1	2	3	5	--	--	--	--	--	--	--	--
200	N/A	1	2	3	6	10	--	--	--	--	--	--
300	N/A	N/A	2	3	5	7	10	15	--	--	--	--
400	N/A	N/A	N/A	3	5	7	9	12	15	20	--	--
500	N/A	N/A	N/A	N/A	5	7	9	12	14	17	20	25
600	N/A	N/A	N/A	N/A	5	7	9	11	14	16	19	22
700	N/A	N/A	N/A	N/A	N/A	7	9	11	13	16	18	21
800	N/A	N/A	N/A	N/A	N/A	6	9	11	13	16	18	21
900	N/A	N/A	N/A	N/A	N/A	N/A	8	11	13	15	18	20
1000	N/A	N/A	N/A	N/A	N/A	N/A	8	11	13	15	17	20

If certain expansion anchors are not accessible due to such things as high radiation, concrete poured over the anchorage, equipment disassembly or removal being required, etc., then other methods may be used to assess the tightness of the expansion anchors.

- Use the Reduced Inspection Alternative (Section 6.3.9.2) to evaluate the anchorage adequacy (the reduced inspection does not require a tightness check).
- Delay the tightness checks until radiation hazards are less.
- Use engineering judgment to assess the anchorage adequacy based on other considerations, e.g., tightness checks on similar anchors elsewhere in the facility which show that installation practices produced consistently tight installation. This method should be used as a last resort. The basis for the engineering judgment should be documented.

6.3.9.2 Reduced Inspection Procedure for Expansion Anchors[42]

A reduced level of inspection can be performed for expansion anchors if additional conservatism is included in the anchorage evaluation. The two inspections which can be deleted for this reduced inspection are:

- Tightness Check (Section 6.3.9.1)
- Embedment Check (Section 6.3.3)

[42] Based on Section C.2.10 of SQUG GIP (Ref. 1)

However to use this Reduced Inspection Alternative, the following conditions should be met:

- <u>Capacity Reduction Factor Applied</u>. If the Reduced Inspection Alternative is used, then a pullout capacity reduction factor (RI_p) and shear capacity reduction factor (RI_s) should be multiplied by the nominal pullout and shear capacities (P_{nom}, V_{nom}) given in Table 6.3-1.

  RI_p = Pullout capacity reduction factor for use with Reduced Inspection Alternative

  = 0.75

  RI_s = Shear capacity reduction factor for use with Reduced Inspection Alternative

  = 0.75

- <u>Other Effects Do Not Reduce Anchor Capacity</u>. None of the other effects which could lower the capacity of the anchor are present. The following anchorage inspection checks, should show that the anchors have full capacity. The checks and the full capacity values are listed:

  Gap Size: None (Section 6.2.3)

  Spacing: $S \geq 10D$ (Section 6.3.4.1)

  Edge Distance: $E \geq 10D$ (Section 6.3.5.1)

  Concrete Strength:

  For Pullout: $f_c' \geq 4000$ psi (Section 6.3.6.1)

  For Shear: $f_c' \geq 3500$ psi (Section 6.3.6.1)

  Concrete Cracks: None (Section 6.3.7.1)

  Essential Relays: None (Section 6.3.8)

- <u>One Third of Anchors Not Available</u>. The applied seismic and dead loads should be less than the allowable anchor pullout and shear capacities given above when a third of the anchors securing the item of equipment are assumed to be unavailable for carrying loads, i.e., 50% more bolts are used to secure the item of equipment than necessary to meet the allowable loads. There should be at least six anchors securing the equipment with four assumed to be carrying the load and two not.

6.3.10 Other Anchor Types

6.3.10.1 Welds to Embedded Steel or Exposed Steel[43]

Equipment at DOE facilities are often anchored by welds to steel plates or channels which are embedded in concrete (see Figure 6.3-1). The strength of such an anchorage depends on the weld of the equipment to the steel and the shear and pullout resistance of the headed stud that anchors the steel into the concrete. The following topics are covered in this section:

- Allowable Loads for Typical Welds (Section 6.3.10.1.1)
- Summary of Equivalent Weld Sizes (Section 6.3.10.1.2)
- Weld Check (Section 6.3.10.1.3)
- Embedded or Exposed Steel Check (Section 6.3.10.1.4)

The specific checks described in this section should be performed in conjunction with the generic anchorage installation inspection checks described in the rest of Section 6.2.

6.3.10.1.1 Allowable Loads for Typical Welds[44]

The allowable loads for typical welds made with E60 electrodes are listed in Table 6.3-13. These allowable loads are based on a weld stress allowable of 30,600 psi.

Table 6.3-13 Allowable Capacities for Typical Welds (E60 Electrodes) (Table C.6-1 of SQUG GIP, Ref. 1)

Weld Sizes		Throat Area	Allowable
t (in.)	L (in.)	(A = .707 t L) ($in.^2$)	F_w (kips)
1/8	1/2	0.0442	1.35
1/8	3/4	0.0663	2.03
1/8	1	0.0884	2.70
3/16	1/4	0.0331	1.01
3/16	1/2	0.0663	2.03
3/16	3/4	0.0994	3.04
3/16	1	0.1326	4.06
1/4	1/4	0.0442	1.35
1/4	1/2	0.0884	2.70
1/4	3/4	0.1326	4.06
1/4	1	0.1768	5.41

43 Based on Section C.6 of SQUG GIP (Ref. 1)
44 Based on Section C.6.1 of SQUG GIP (Ref. 1)

Where:			
	t	=	Thickness of the weld leg
	L	=	Length of the weld
	A	=	Cross-sectional area through the throat of the weld
		=	$0.707\ t\ L$
	F_w	=	Allowable load capacity of weld

6.3.10.1.2 Summary of Equivalent Weld Sizes[45]

A summary of equivalent weld sizes which have the same capacity as other types of fasteners is shown in Table 6.3-14.

Table 6.3-14 Summary of Equivalent Weld Sizes (Table C.6-2 of SQUG GIP, Ref. 1)

Welds		Equivalent Bolt Diameter (D, in.)	
Typical Size (L x t, in.)	Throat Area ($in.^2$)	Expansion Anchor Bolts	Cast-in-Place Anchor Bolts
1/2 x 1/8	0.0442	3/8	--
1 x 1/8	0.0884	1/2	--
1 x 3/16	0.1326	3/4	3/8
1 x 1/4	0.1768	3/4	3/8
2 x 3/16	0.2651	7/8	1/2
2 x 1/4	0.3535	1	5/8
2 x 3/8	0.5305	--	3/4

[45] Based on Section C.6.2 of SQUG GIP (Ref. 1)

6.3.10.1.3 Weld Check[46]

The welds used for anchoring equipment to embedded or exposed steel should be inspected in the following areas:

- Determine the overall length (L) and thickness (t) of the welds. The weld thickness should be limited to the thinnest part of either the weld itself or the connecting part.
- Check for weld burn-through on cabinets made of thin material.
- Check for weld quality, particularly in puddle welds which carry high tension loads.
- The minimum effective length of fillet welds should not be less than 4 times the nominal size of the weld, or else the size of the weld should be considered not to exceed 1/4 of its effective length.

6.3.10.1.4 Embedded or Exposed Steel Check[47]

The embedded steel or the exposed steel to which the equipment is anchored by the weld should be evaluated to determine whether it has the capacity to carry the loads applied to it.

The allowable stresses from Part 2 of the AISC code (Ref. 81) may be used for evaluating the adequacy of exposed steel and the structural members of an embedded steel assembly. The guidelines given in Section 6.3 can be used for evaluating the cast-in-place bolts and headed studs which are a part of the embedded steel assembly.

[46] Based on Section C.6.3 of SQUG GIP (Ref. 1)
[47] Based on Section C.6.5 of SQUG GIP (Ref. 1)

6.3.10.2 Lead Cinch Anchors

This section is adapted from Section 4.3 of Part III of SEP-6, Revision 1, "The Procedure for the Seismic Evaluation of SRS Systems using Experience Data" (Ref. 3), which was developed for the Savannah River Site (SRS).

Nominal allowable capacities for lead anchors are given in Table 6.3-15. These values are derived from SRS in-situ test data (Ref. 3) with a factor of safety of at least 4. The derivation of allowables for lead anchors is consistent with the anchorage methodology of the DOE Seismic Evaluation Procedure.

Table 6.3-15 Allowable Loads for Inspected Lead Anchors (Table C-1 of Ref. 3)

Bolt Diameter (in.)	Allowable Tension (lbs.)	Allowable Shear (lbs.)
3/8	600	400
1/2	870	800
5/8	970	1,400
3/4	1,280	2,000
1	3,160	3,500

The above allowables are to be used for all lead anchors that have been successfully inspected. Higher tension allowables may be used if the bolt can be torqued to induce the desired tension load. Figures 6.3-7 to 6.3-10 give the 95% lower confidence bound torque tension correlation needed to evaluate the proof torque. Note that these curves cannot be extrapolated to give higher allowables. Following the additional torque check, the gap must be re-evaluated between the top of the shell and the bottom of the equipment base.

These allowables are applicable if the minimum criteria for bolt to bolt spacing (10D) and bolt to edge distance (10D) are satisfied, and installation adequacy is assured. When the edge distance and bolt-to-bolt requirements are not met, the allowables can be reduced as for expansion anchors (see Sections 6.3.4.1, 6.3.5.1, 6.3.6.1, 6.3.7.1, and 6.3.8).

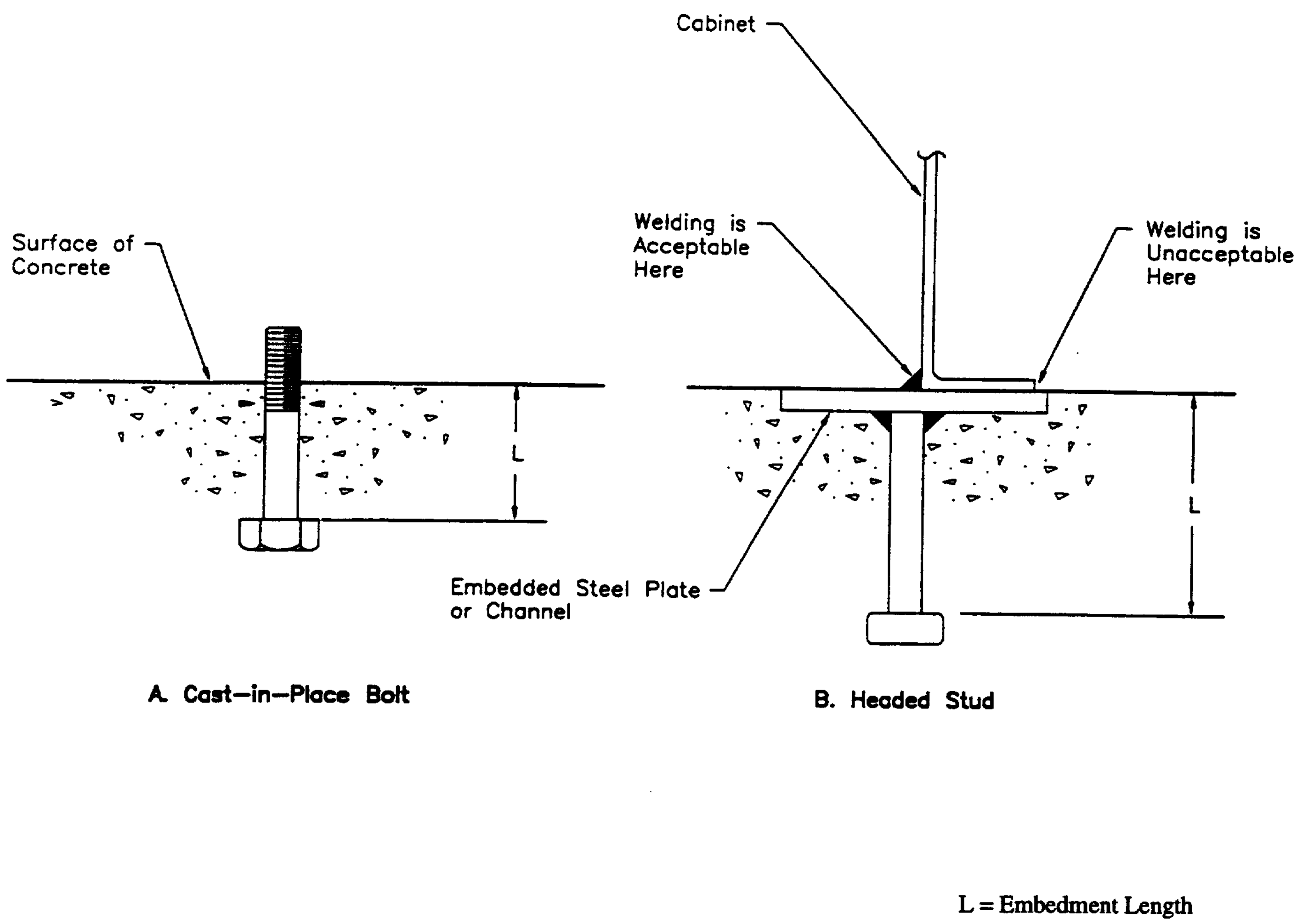
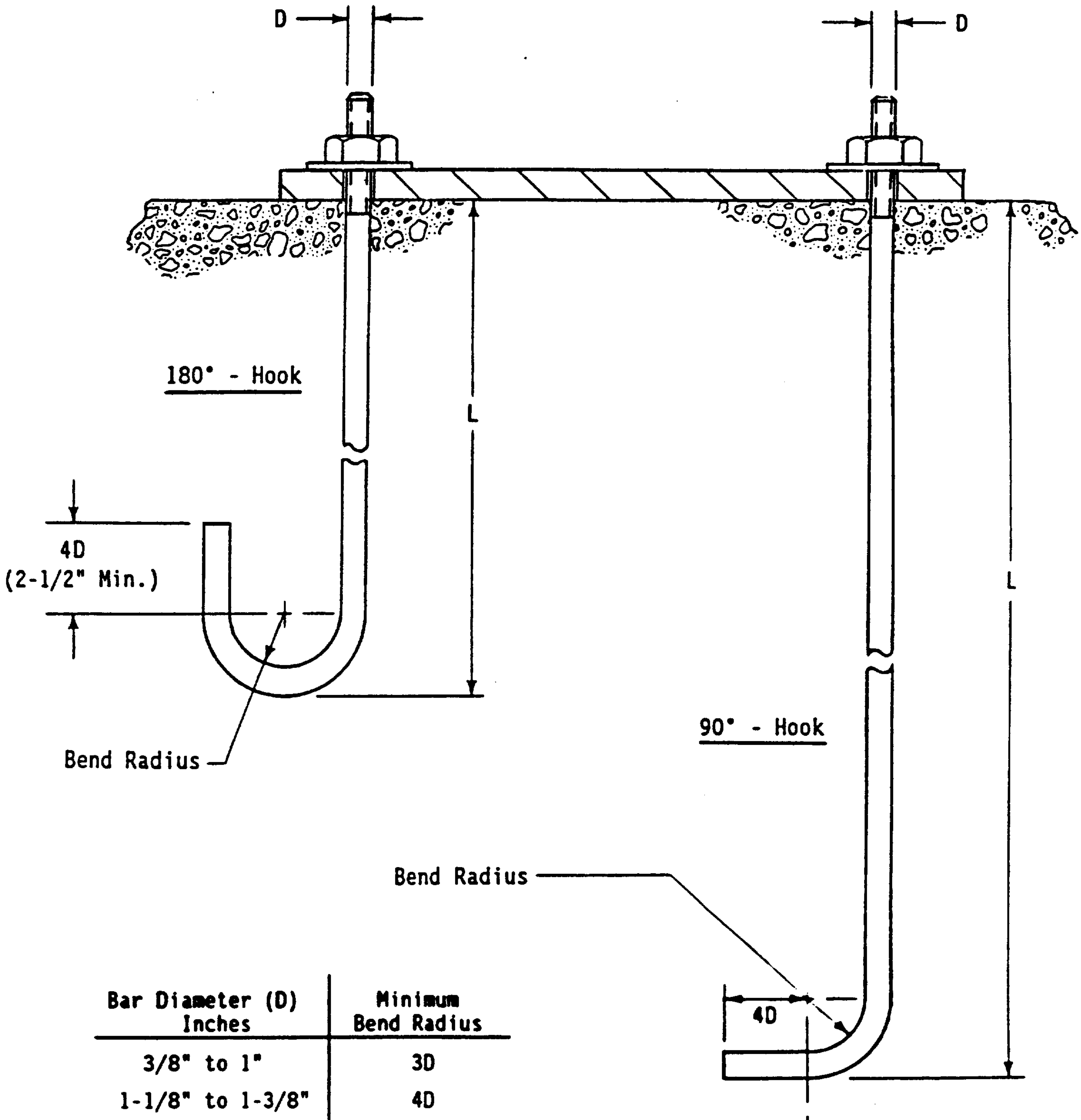


Figure 6.3-1 Typical Installations of Cast-In-Place Bolt and Headed Stud (Figure C.3-1 of SQUG GIP, Reference 1)



Bar Diameter (D) Inches	Minimum Bend Radius
3/8" to 1"	3D
1-1/8" to 1-3/8"	4D

Figure 6.3-2 Typical J-Bolt Installations (Reference 41) (Figure C.4-1 of SQUG GIP, Reference 1)

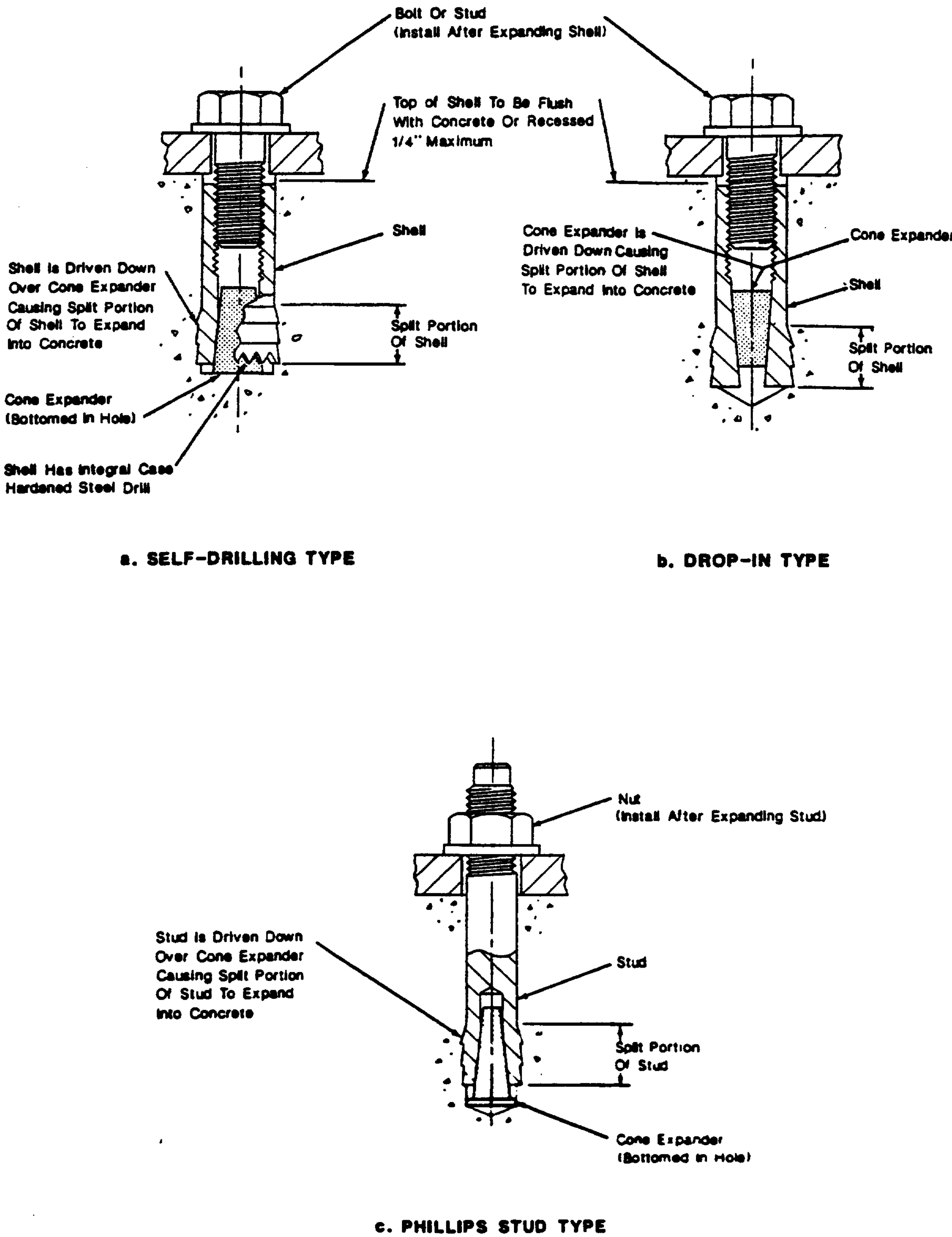
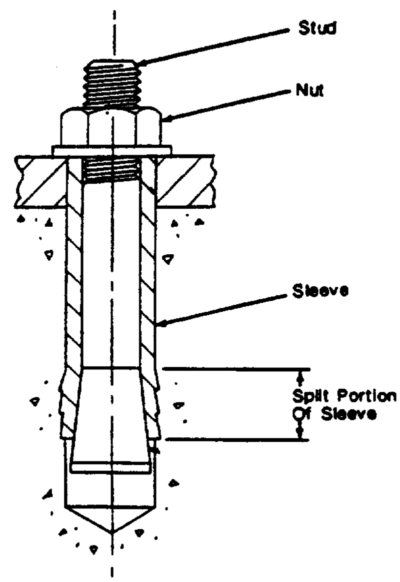
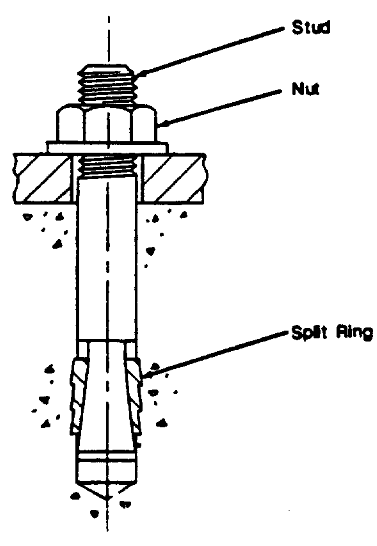


Figure 6.3-3 Features of Shell-Type Expansion Anchors (Reference 41) (Figure C.2-1 of SQUG GIP, Reference 1)



a. SLEEVE TYPE



b. WEDGE TYPE

Figure 6.3-4 Features of Nonshell-Type Expansion Anchors (Reference 41) (Figure C.2-2 of SQUG GIP, Reference 1)

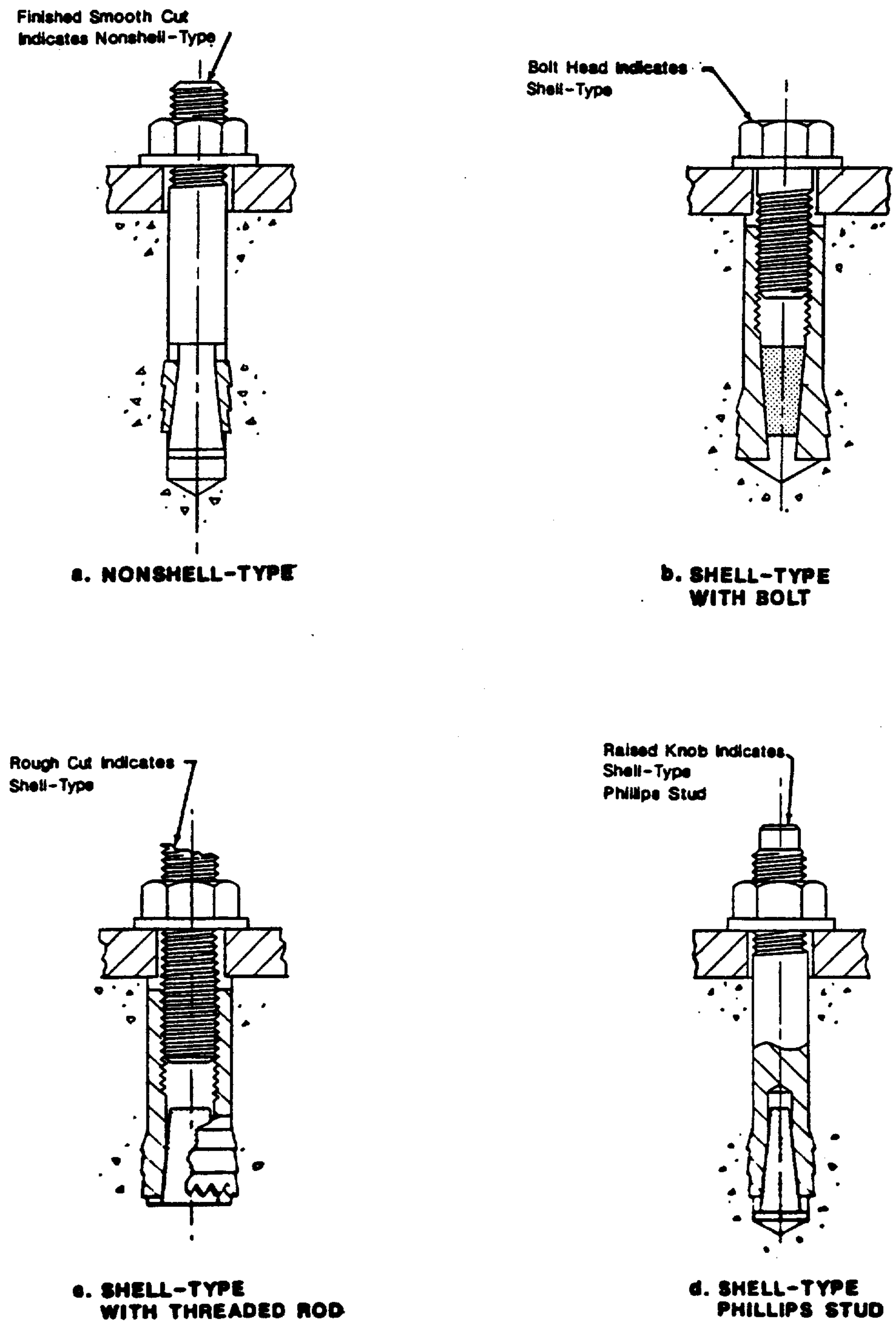


Figure 6.3-5 Distinguishing Characteristics of Installed Shell- and Nonshell-Type Expansion Anchors (Reference 41) (Figure C.2-3 of SQUG GIP, Reference 1)

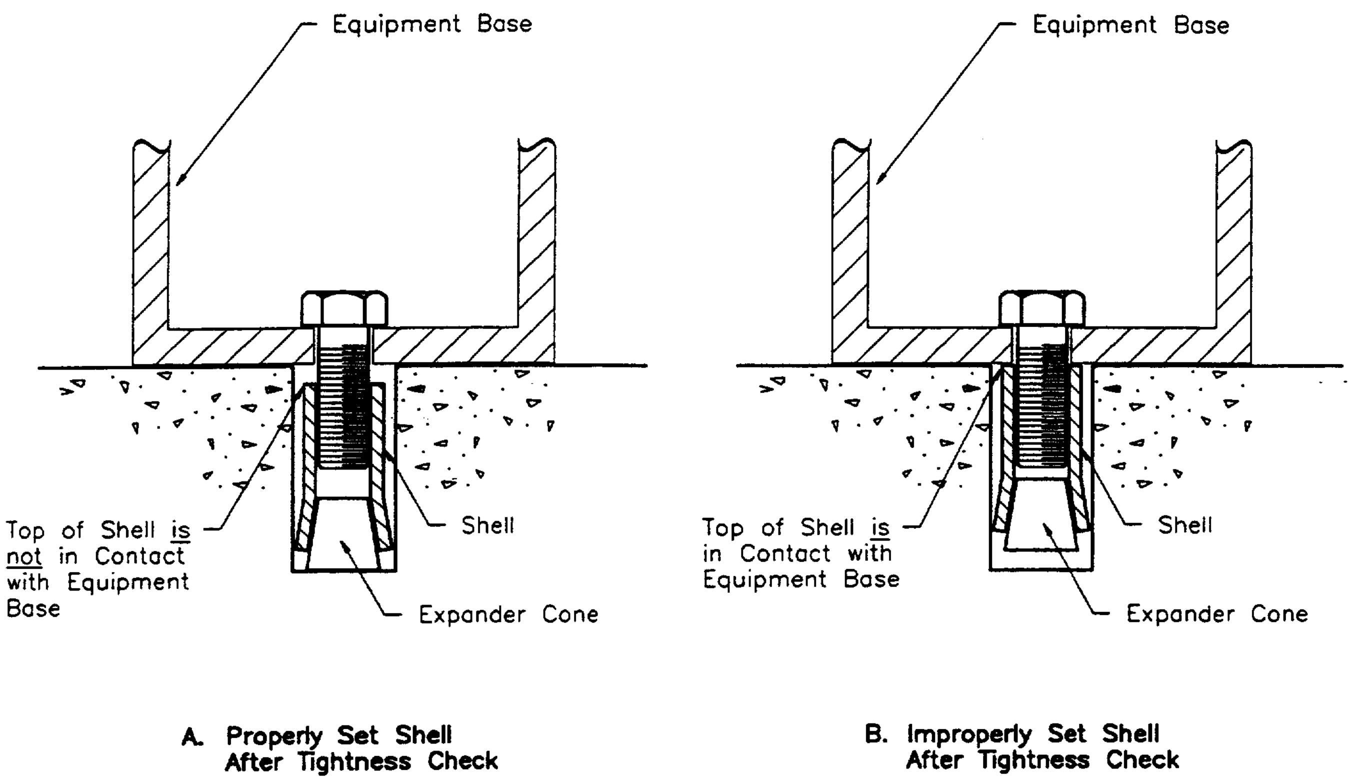


Figure 6.3-6 Installation of Shell-Type Expansion Anchors (Figure 4-4 of SQUG GIP, Reference 1)

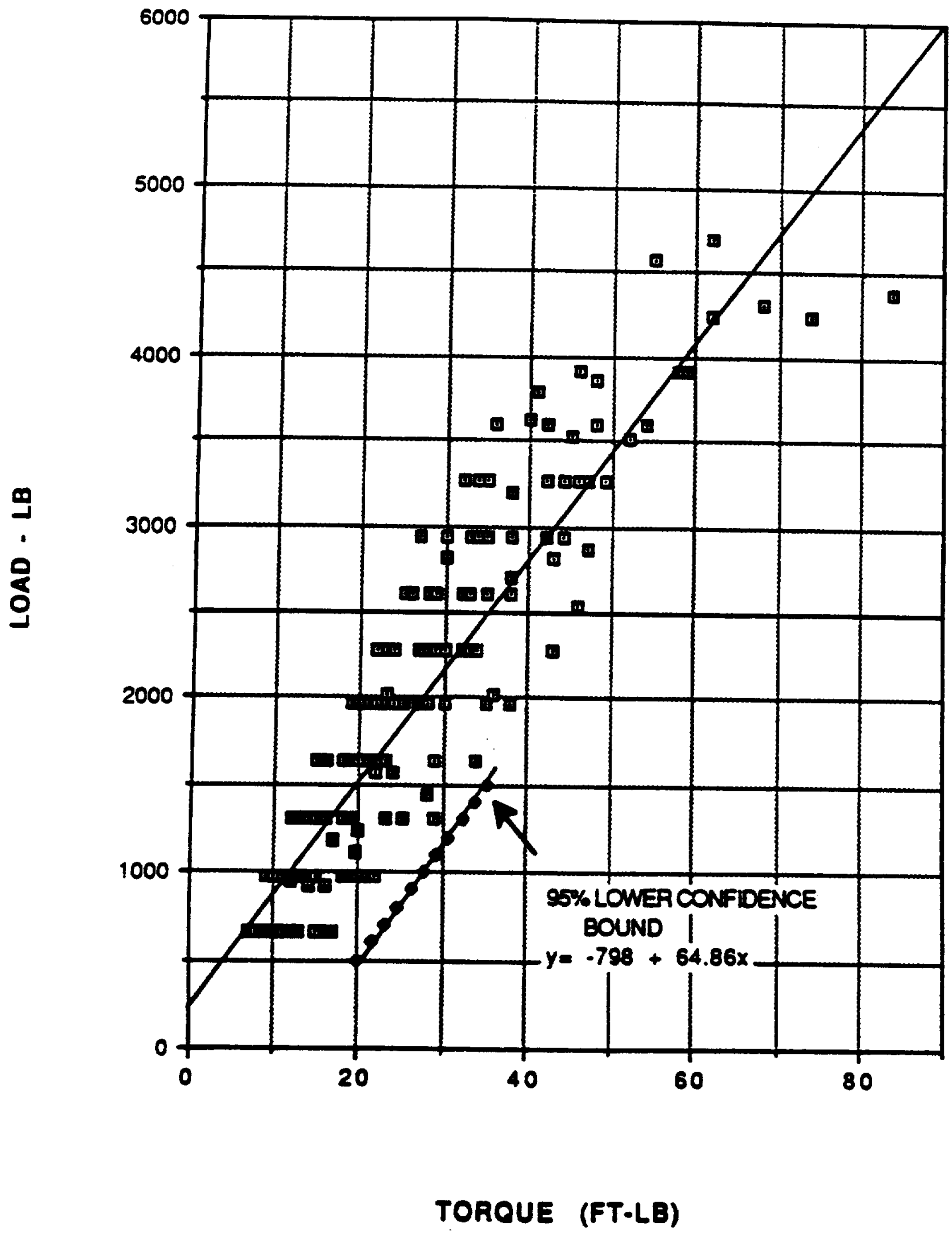


Figure 6.3-7 Torque vs Load Tension Test for 3/8" Anchor (Figure C-1 of WSRC SEP-6, Reference 3)

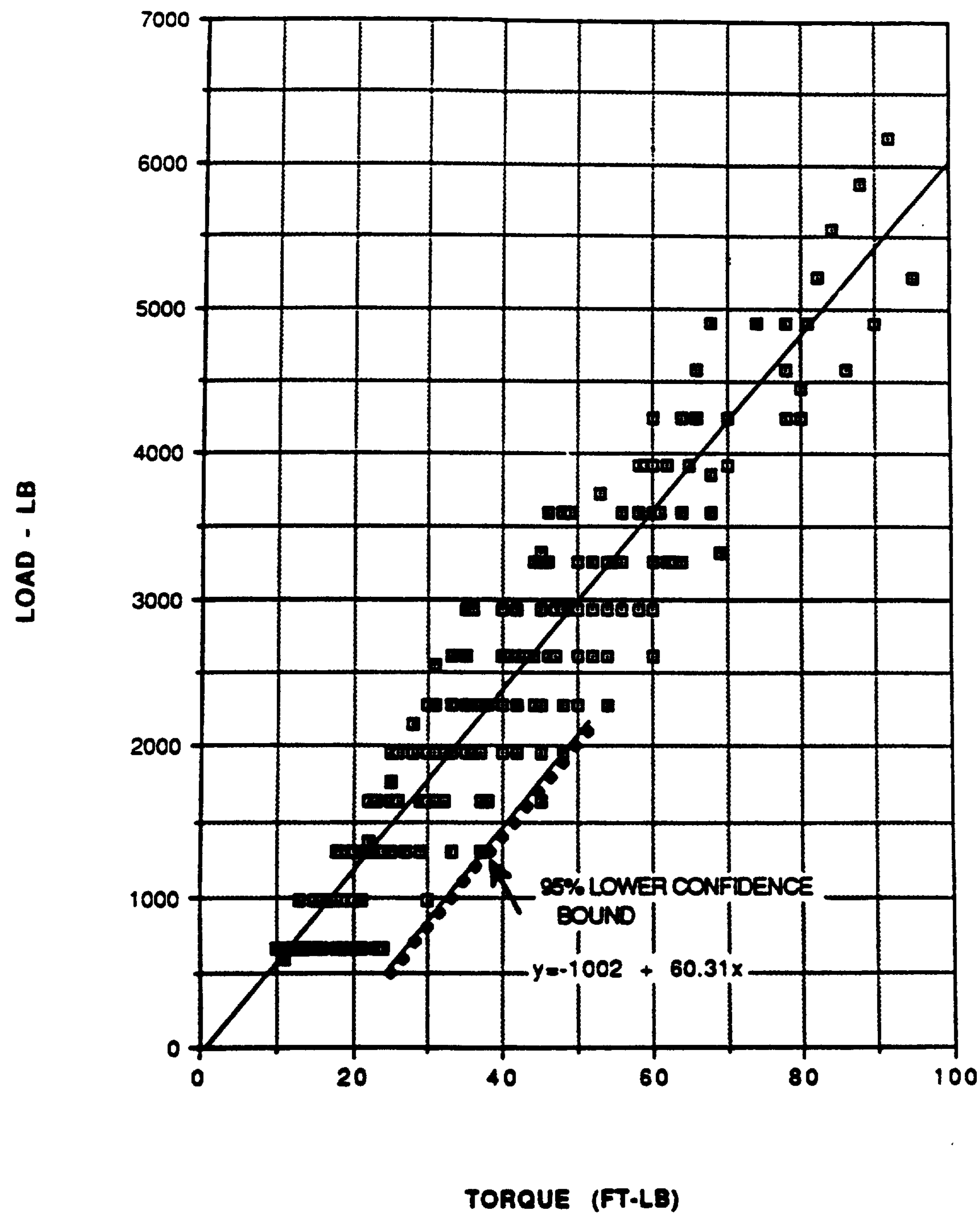


Figure 6.3-8 Torque vs Load Tension Tests for 1/2" Anchor (Figure C-2 of WSRC SEP-6, Reference 3)

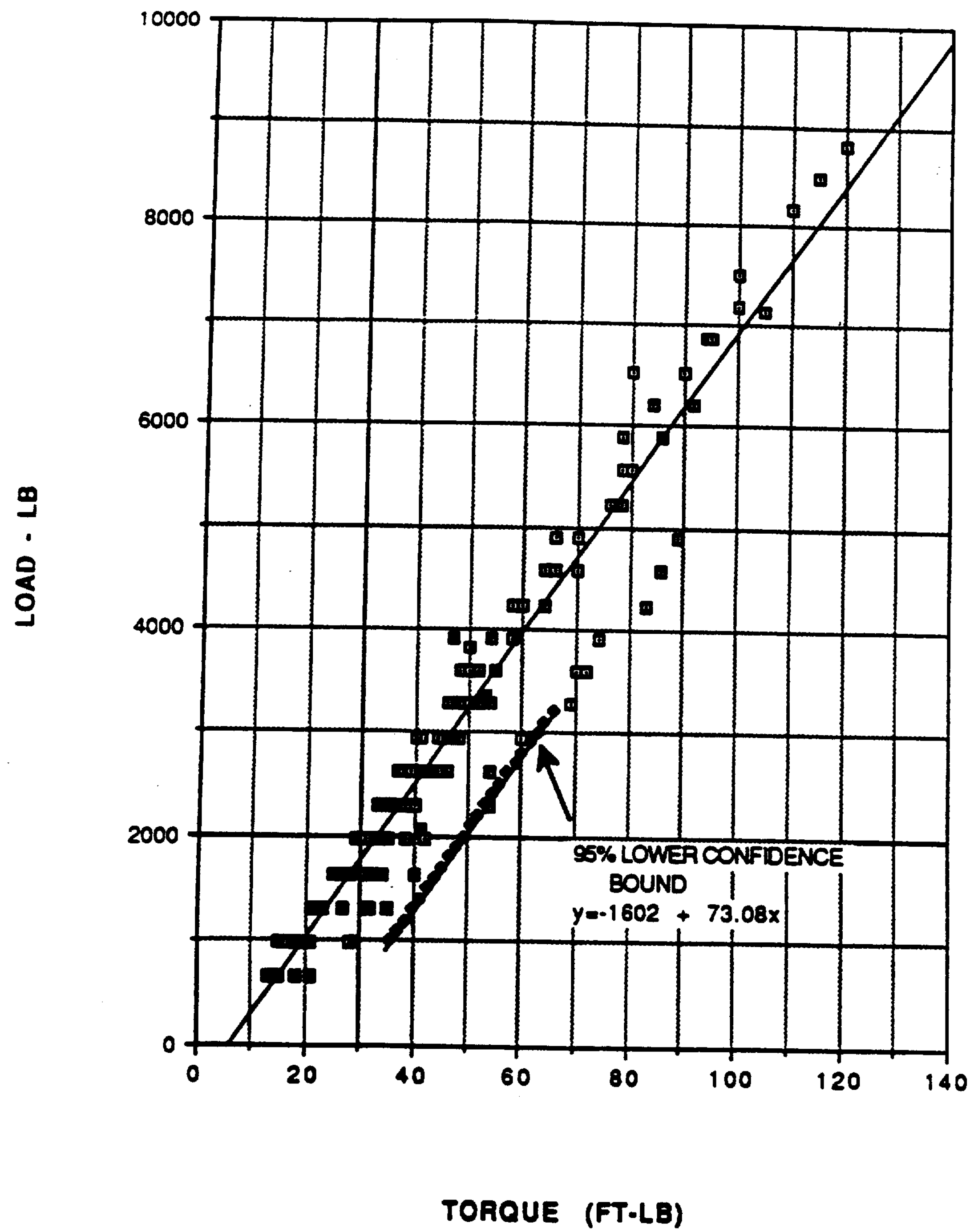


Figure 6.3-9 **Torque vs Load Tension Test for 5/8" Anchor (Figure C-3 of WSRC SEP-6, Reference 3)**

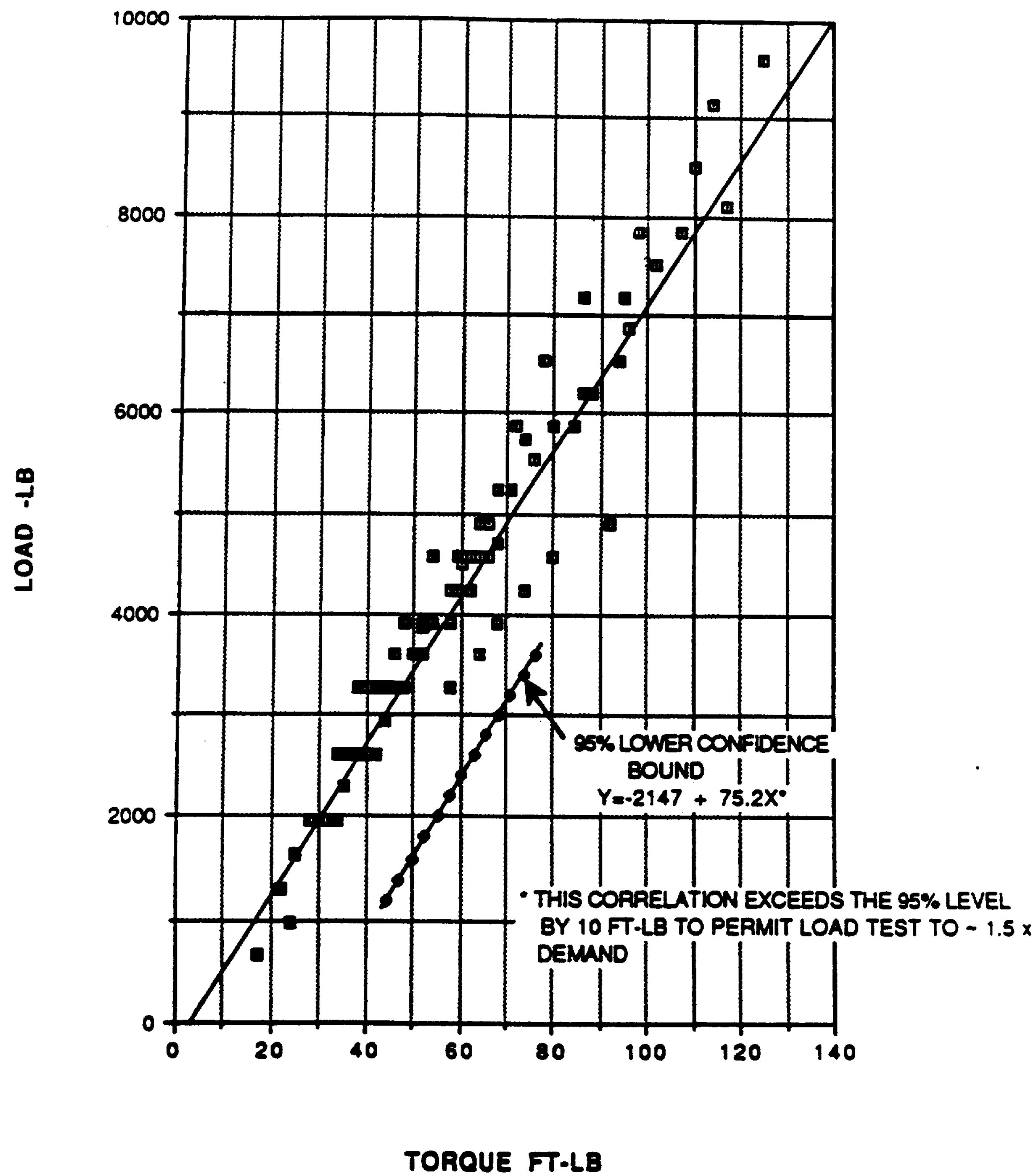


Figure 6.3-10 Torque vs Load Tension Test for 3/4" Anchor (Figure C-4 of WSRC SEP-6, Reference 3)

6.4 ANCHORAGE DEMAND DETERMINATION

6.4.1 Equipment Characteristics[48]

To determine the seismic demand on the anchorage of an item of equipment, the following equipment characteristics should be estimated: mass, location of the center of gravity, natural frequency, component damping, and equipment base center of rotation for overturning moment.

The mass of the equipment is a primary parameter for determining the inertial loads applied to the anchorage. Equipment weight can be obtained from drawings and/or original purchase documents, if available. However, if this information is not available, then conservative estimates of equipment weight for several equipment classes are discussed below. These estimated masses are, in general, based on the heaviest (or most dense) item identified during a survey of typical equipment in each of the equipment classes. For unusual equipment, an independent mass calculation should be performed or a conservative estimate made.

The location of the center of gravity of the equipment is used to determine the overturning moment caused by the inertial loads. It should be estimated by performing a visual inspection of the equipment. If the equipment has relatively uniform density, the center of gravity can be taken at the geometric center of the equipment. If the mass of the equipment is skewed, then appropriate adjustments should be made to the center of gravity location. If the equipment mass is centered significantly offset from the geometric centerline, then this should be noted and torsional effects included in the anchorage evaluations.

The lowest natural frequency (f_n) of the equipment is used to determine the amplified acceleration of the equipment from the in-structure response spectrum. Only the overall structural modes of the equipment need be considered for anchorage evaluations. Since equipment-specific information is normally not available for determining the natural frequency of most types of equipment, approximate natural frequencies for certain classes of equipment are discussed below as either rigid ($f_n >$ about 20 Hz) or flexible ($f_n <$ about 20 Hz). Reference 77 also contains guidance for estimating the natural frequency of equipment.

The equipment damping should be determined for flexible equipment so that an appropriate in-structure response spectrum, with the appropriate level of damping, is used to obtain spectral accelerations. The damping values for certain classes of equipment are discussed below.

The center of rotation of the equipment base is the line on the base about which the equipment would rotate due to an overturning moment. The location of the center of rotation should be estimated based on the following guidance. For very rigid equipment bases, such as heavy machinery on skid mounts, the equipment may be considered to pivot about its outer edge or far side bolt centerline. For flexible equipment bases, such as electrical cabinets with light base framing members, the center of rotation should be taken close to the equipment base centerline.

This remainder of this section contains estimates of equipment mass, natural frequency, and damping for the various classes of equipment for anchorage evaluations as summarized in Table 6.4-1. For those classes of equipment not covered in Table 6.4-1, the relative flexibility/stiffness and damping should be estimated based on engineering judgment, past experience, and comparison to the equipment provided in Table 6.4-1.

The purpose of Table 6.4-1 is to describe generic characteristics which may be used during anchorage evaluations in place of equipment-specific data. These generic characteristics typically

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[48] Based on Sections 4.4.1 - Check 1 and C.1 of SQUG GIP (Ref. 1)

result in larger than actual loadings on the anchorage. However, for unusual items of equipment, e.g., motor control center weighing 800 pounds with an additional 100 pounds external weight, an independent check should be made of the reasonableness of the values contained in Table 6.4-1.

The equipment mass contained in Table 6.4-1 is based on the heaviest item found in each of the classes covered during a survey of equipment. Note that these masses are the same as those used in the screening tables given in the EPRI Anchorage Report (Ref. 41) except for the motor control centers which use 625 pounds per cabinet in the screening tables instead of the 800 pounds given in Table 6.4-1.

Equipment lowest natural frequency is given as a relative rigidity of either "rigid" or "flexible" in Table 6.4-1. Equipment with a lowest natural frequency of the overall structural mode greater than about 20 Hz is considered rigid. Equipment with natural frequencies below about 20 Hz are considered flexible. Note that "rigid" and "flexible" categories of equipment in Table 6.4-1 apply only to anchorage evaluations.

The relative rigidities given in Table 6.4-1 are for "typical" equipment in DOE facilities. These generic categories of rigid or flexible should be checked when performing the seismic evaluation, noting particularly the rigidity or flexibility of the base support system for the equipment and the rigidity of the anchorage itself. In particular, the estimate for natural frequency of equipment secured with expansion anchors should take into account the potential for slippage of these types of anchors. This would be necessary, for example, when natural frequency estimates of equipment secured with expansion anchors are based on analytical models which used fixed anchor points or when shake table test results are used in which the equipment was welded to the table.

Data for in-line equipment is not contained in Table 6.4-1.

Figure 6.4-1 provides equations for computing the lowest natural frequency of typical structural frames.

For rigid equipment, the seismic demand on the equipment can be determined by using the Zero Period Acceleration (ZPA) of the appropriate floor response spectrum. For flexible equipment, the peak of the floor response spectrum (for the damping value given in Table 6.4-1) should be used.

Table 6.4-1 Generic Equipment Characteristics for Anchorage Evaluations (Table C.1-1 of SQUG GIP, Ref. 1)

Equipment Class	Typical Maximum Weight or Weight Density		Typical Natural Frequency[b] and Damping
Motor Control Centers (Section 8.1.2)	800 lb per cabinet[d]		Flexible 5% Damping
Low-Voltage Switchgear (Section 8.1.3)	35 lb/ft^3		Flexible 5% Damping
Medium-Voltage Switchgear[a] (Section 8.1.4)	31 lb/ft^3		Flexible 5% Damping
Transformers (Section 8.1.6)	<u>Rating (KVA)</u> 3,000 2,500 2,000 1,000 100	<u>Weight (lb)</u> 15,000 11,050 9,400 6,300 975	Flexible 5% Damping
Horizontal Pumps with Motors (Section 8.2.3)	<u>Power (HP)</u> 1,000 600 500 400 200 100	<u>Weight (lb)</u> 20,000 16,500 12,000 8,600 6,000 3,600	Rigid 5% Damping[c]
Vertical Pumps with Motors (Section 8.2.4)	<u>Power (HP)</u>	<u>Weight (lb)</u>	
a. Vertical Immersion	150	4,000	Flexible 3% Damping
b. Centrifugal	500 2,000	9,000 48,000	Rigid 5% Damping[c]
c. Deep-Well	500	9,000 (motor) 14,000 (pump)	Flexible 3% Damping
Air Compressors (Section 8.2.6)	<u>Power (HP)</u> 50 200	<u>Weight (lb)</u> 4,000 10,000	Rigid 5% Damping[c]

Table 6.4-1 (Continued)

Equipment Class	Typical Maximum Weight or Weight Density	Typical Natural Frequency[b] and Damping
Motor-Generators (Section 8.2.7)	(Not Available)	Rigid 5% Damping[c]
Batteries on Racks (Section 8.1.1)	0.11 lb/in^3 for batteries, plus weight of racks	Flexible 5% Damping
Battery Chargers and Inverters (Section 8.1.7)	45 lb/ft^3	Flexible 5% Damping
Engine-Generators (Section 8.2.8)	(Not Available)	Rigid 5% Damping[c]
Instrument Racks (Section 8.1.9)	20 lb/ft^2 of vertical face	Flexible 3% Damping
Generic Equipment Cabinets (Section 8.1.5)	3 times the weight of cabinet housing	Flexible 5% Damping
Walk-Through Control Panels (Section 8.1.8)	Determine and use weight per foot of length	Flexible 5% Damping

(a) Medium voltage switchgear are called "Metal-Clad Switchgear" in Reference 41.

(b) The lowest natural frequencies of the overall structural mode are given as either Rigid (> about 20 Hz) or Flexible (< about 20 Hz) and apply only to anchorage evaluations.

(c) A damping value of 5% can be used for rigid equipment since the seismic accelerations can be taken from the ZPA which is not affected significantly by damping level.

(d) Note: When using the screening tables in the EPRI Anchorage Report (Reference 41), an average weight per MCC section of 625 pounds was used rather than the 800 pounds shown in this table.

6.4.2 Seismic Loads[49]

The next step in evaluating the seismic adequacy of anchorage is to determine the loads applied to the anchorage by the seismic demand imposed on the item of equipment. This is done using the following five steps:

1. Determine the appropriate input seismic accelerations for the item of equipment for each of the three directions of motion.

2. Determine the seismic inertial equipment loads for each of the three directions of motion using the equivalent static load method.

[49] Based on Section 4.4.3 of SQUG GIP (Ref. 1)

3. Determine the seismic inertial anchor loads by calculating the various load components for each direction of motion.

4. Calculate the combined seismic loads on each anchor from each of the three directions of seismic motion. Then combine the load components from these three directions using the Square Root Sum of the Squares (SRSS) method.

5. Calculate the total anchor loads on each anchor by adding the combined seismic loads to the equipment deadweight loads and any other loads on the equipment.

These five steps are described below:

Step 1 - Input Seismic Accelerations. The first step in determining the seismic demand loads on the anchorage is to compute the input seismic accelerations from an appropriate in-structure response spectrum, at the damping and natural frequency of the equipment, for the location in the facility where the equipment is mounted. Section 5.2.2 discusses the techniques for determining the scaled in-structure response spectrum (SDS) which is computed from the Design Basis Earthquake (DBE).

If the equipment is located in an area where there are two applicable lateral response spectra (nominally one N-S and one E-W), then one of the following alternatives can be used to define a single horizontal seismic demand acceleration for load calculation:

- Use the higher acceleration for both horizontal directions.
- Use the acceleration value (either N-S or E-W) which aligns with the direction of the "weak" anchorage for that item of equipment.
- Use the actual direction N-S and E-W accelerations for the N-S and E-W loads on each item of equipment.

The vertical component of acceleration should be the appropriate site-specific fraction of the horizontal component of acceleration. For most equipment classes, the vertical direction fundamental frequency is in the rigid range.

The following factors which should be considered in determining the input seismic accelerations are covered below: equipment damping, natural frequency of the equipment, and use of unbroadened response spectra.

Equipment Damping. A 5% damping value can be used in anchorage evaluations for most of the equipment classes covered by this procedure. Section 6.4.1 lists the equipment classes for which 5% damping is recommended. This level of damping is adequate for these classes because the equipment either exhibits this level of damping or it is essentially rigid (natural frequency greater than about 20 Hz) so that the damping level is nearly irrelevant. Section 6.4.1 also lists the classes of equipment which have lower damping (3% damping) and which are, in general, flexible. This equipment includes electrical equipment and some types of Vertical Pumps. It should be evaluated that the equipment does not have unusual features which could lower its damping below the values given in Section 6.4.1.

In-structure response spectra for the facility may not be available at the 5% or 3% damping levels recommended in this procedure for anchorage evaluations. Therefore available response spectra may be normalized to the desired spectral damping level using one of the methods from Appendix A of Reference 19.

For in-structure response spectra which have a shape similar to the Reference Spectrum, (without very narrow peaks) the spectral acceleration for a desired damping ratio β_D can be estimated from an available response spectrum with a damping ratio of β_A by using the following relationship:

$$Sa_{iD} = Sa_{iA} \sqrt{\frac{\beta_A}{\beta_D}}$$

However this spectral acceleration Sa_{iD} is limited to:

$$Sa_{iD} \geq ZPA$$

for frequencies (f_i) in the high frequency region; i.e. frequencies greater than the frequency associated with the peak of the response spectrum.

The meaning of the symbols used above is as follows:

Sa_{iA} = available spectral acceleration at frequency f_i associated with a damping ratio β_A

Sa_{iD} = desired spectral acceleration at frequency f_i associated with a damping ratio β_D

β_A = damping ratio of available response spectrum

β_D = damping ratio of desired response spectrum

ZPA = Zero Period Acceleration

f_i = frequency of interest

Natural Frequency. The lowest natural frequency (f_n) of the equipment may be estimated by past experience with testing or analysis. The natural frequency of the equipment can be determined during the inspection of the anchorage installation. Note that reasonable estimates of equipment natural frequency for several equipment classes are given in Section 6.4.1 as either rigid (f_n > about 20 Hz) or flexible (f_n < about 20 Hz). The following classes of equipment can generally be considered as rigid (i.e., natural frequency greater than about 20 Hz) if anchored stiffly:

- Horizontal Pump (Section 8.2.3)
- Air Compressors (Section 8.2.6)
- Motor-Generators (Section 8.2.7)
- Engine-Generators (Section 8.2.8)

Rigid equipment can use a damping value of 5% since it is not significantly amplified over the Zero Period Acceleration (ZPA).

If the natural frequency of the equipment is estimated to be high (i.e., greater than about 20 Hz), then the equipment should be considered "rigid" and the Zero Period Acceleration (ZPA) should be used for anchorage load calculations. If the natural frequency is estimated to be below about 20 Hz, then the equipment should be considered "flexible" and the peak of the response spectrum may conservatively be used for anchorage load calculations. If the natural frequency of the equipment is known (by calculation, test, or other means), the maximum acceleration from the response spectrum for the frequency range of interest (from equipment natural frequency to 33 Hz) can be used instead of the peak.

Unbroadened Response Spectra. Unbroadened in-structure response spectra can be used for comparison to seismic capacity spectra. Uncertainty in the natural frequency of the building structure should be addressed by shifting the frequency of the seismic demand response spectrum at these peaks. A reference or basis for establishing the degree of uncertainty in the natural frequency of the building structure should be included in the facility-specific seismic evaluation records. The method of peak shifting discussed in ASCE 4 (Ref. 74) may also be used.

Step 2 - Seismic Inertial Equipment Loads. The second step in determining the seismic demand loads on the anchorage is to compute the seismic inertial equipment loads for each of the three directions of motion using the equivalent static load method. In this method, the seismic analysis is performed statically by applying the inertial load at the center of gravity of the equipment. The inertial load in each direction is equal to the product of the input seismic accelerations, an equivalent static coefficient, and the mass of the equipment.

An equivalent static coefficient of 1.0 can be used for the classes of equipment covered by this procedure; the basis for this is described in Reference 41. The mass of the equipment is determined during the inspection of the anchorage installation. Note that conservative estimates of equipment mass for several equipment classes are given in Section 6.4.1.

Step 3 - Seismic Inertial Anchor Loads. The third step in determining the seismic demand loads on the anchorage is to compute the seismic inertial anchor loads for each of the three directions of motion. This is done by applying the seismic inertial equipment loads determined in the previous step to the center of gravity of the item of equipment and calculating the free-body loads on the anchors. The location of the center of gravity of the equipment is determined during the inspection of the anchorage installation. The location of the center of gravity can be taken as the geometric center of the equipment if the equipment is of uniform density. If the mass of the equipment is skewed, then appropriate adjustments should be made to the center of gravity location.

The following types of seismic inertial anchor loads should be determined. Note that these loads are applicable whether the equipment is mounted on the floor, wall, or ceiling.

- Anchor shear loads due to the lateral component of force caused by the seismic inertial equipment loads, including, if significant, the anchor shear loads due to any torsional moments (center of gravity is not in line with the centroid of the group of anchors).

- Anchor pullout loads due to the overturning moment caused by the seismic inertial equipment loads, with an appropriately estimated location of the overturning axis. Guidance on estimating the location of the overturning axis is given below.)

- Anchor pullout loads caused by the seismic inertial equipment loads due to the component of force which is in line with the axes of the anchor bolts; e.g., for floor-mounted equipment include the vertical component of the seismic load.

The anchor loads caused by the equipment overturning moment can be based on the assumption that plane sections remain plane during loading and that the material in the equipment and the anchors behave in a linear-elastic manner. This results in a linear distribution of anchor loads for a set of anchors which are equal in stiffness and size.

The recommended location for the overturning axis is at the equipment centerline for equipment with flexible bases. For rigid base equipment, the overturning axes can be taken at the edge of the equipment. Reference 78 contains discussion on locating the overturning axes.

Step 4 - Combined Seismic Loads. The fourth step in determining the seismic demand loads on the anchorage is to compute the combined seismic anchor loads of the seismic loads on each anchor from the three directions of earthquake motion. The combined loads can be computed with a combination technique such as the Square Root Sum of the Squares (SRSS) or the 100-40-40 Rule.

Step 5 - Total Anchor Loads. The total loads on the anchorage are computed by combining the combined seismic anchor loads from the previous step to the equipment deadweight loads and any other significant loads which would be applied to the equipment, e.g.; pipe reaction loads on pumps.

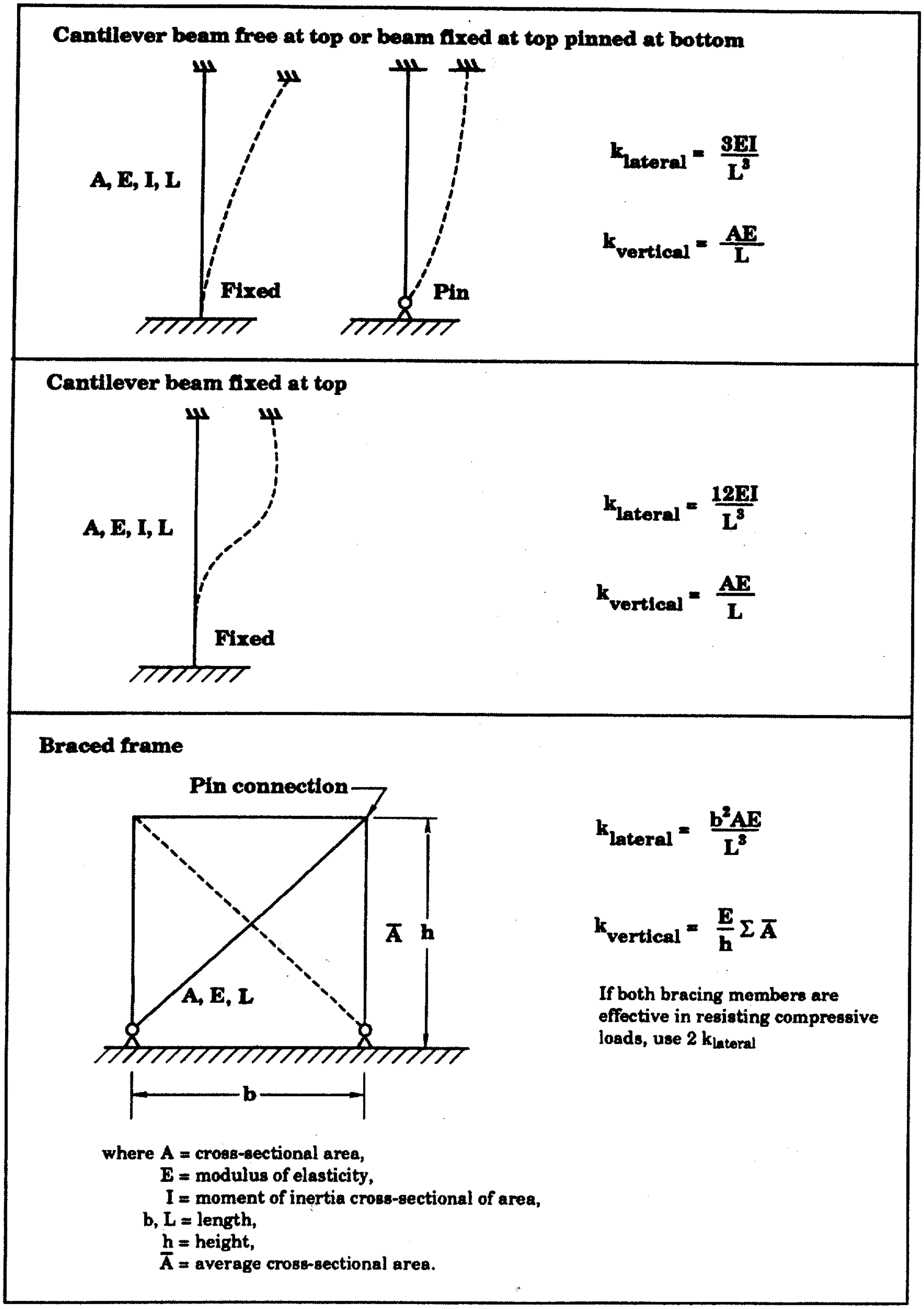

Figure 6.4-1 Stiffness Equations for Structural Frames (Reference 76)

6.5 COMPARISON OF CAPACITY TO DEMAND[50]

The final main step in evaluating the seismic adequacy of anchorage is to compare the seismic capacity loads of the anchors (determined in Section 6.3) to the total anchor loads (determined in Section 6.4). This comparison can be done using the shear-tension interaction formulations given below for each of the anchor types covered by this procedure.

6.5.1 Expansion Anchors[51]

When expansion anchors are subjected to simultaneous shear and tension, one of the following shear-tension interaction formulations should be used. The linear formulation is conservative. The bi-linear formulation is more realistic. Figure 6.5-1 illustrates these formulations.

- Linear Formulation (conservative)

$$\frac{V}{V_{all}} + \frac{P}{P_{all}} \leq 1.0$$

- Bilinear Formulation (more realistic)

$$\frac{P}{P_{all}} \leq 1.0 \qquad \text{for} \quad \frac{V}{V_{all}} \leq 0.3$$

$$0.7\,\frac{P}{P_{all}} + \frac{V}{V_{all}} \leq 1.0 \quad \text{for} \quad 0.3 < \frac{V}{V_{all}} \leq 1.0$$

Where:

Where:	P	=	Applied pullout loads due to earthquake plus dead loads.
	V	=	Applied shear loads due to earthquake plus dead loads.
	P_{all}	=	Allowable pullout capacity load for the anchor.
	V_{all}	=	Allowable shear capacity load for the anchor.

6.5.2 Cast-in-Place Bolts and Headed Studs[52]

For existing cast-in-place bolts subjected to simultaneous shear and tension, the shear-tension interaction depends on the anticipated failure mode. Figure 6.5-2 presents the interaction curves for cast-in-place bolts for failure in the bolt steel or failure in the concrete. Because the anchorage criteria in this procedure and Reference 41 for cast-in-place bolts and headed studs are based on an additional factor of safety of 1.5 against failure not occurring in the concrete, it is recommended that the interaction formulation for steel failure be used.

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50 Based on Section 4.4.4 of SQUG GIP (Ref. 1)

51 Based on Section C.2.11 of SQUG GIP (Ref. 1)

52 Based on Section C.3.7 of SQUG GIP (Ref. 1)

6.5.3 Cast-in-Place J-Bolts[53]

It is left to the user to select an appropriate shear-tension interaction formulation for use with J-bolts when both tension and shear loads are significant.

6.5.4 Grouted-in-Place Bolts[54]

For grouted-in-place bolts subjected to simultaneous shear and tension, the guidelines given in Section 6.5.2 for cast-in-place bolts may be used to compare the allowable loads to the applied loads.

6.5.5 Welds to Embedded Steel or Exposed Steel[55]

When welds are subjected to simultaneous shear and tension, the allowable loads can be compared to the applied loads using the following shear-tension interaction formulation:

$$\left(\frac{P}{F_W}\right)^2 + \left(\frac{V}{F_W}\right)^2 \leq 1$$

Where:

P	=	Pullout (tensile) load applied to weld [kip]
V	=	Shear load applied to weld [kip]
F_w	=	Allowable load for weld from Table 6.3-13 [kip]

53 Based on Section C.4.7 of SQUG GIP (Ref. 1)
54 Based on Section C.5.4 of SQUG GIP (Ref. 1)
55 Based on Section C.6.4 of SQUG GIP (Ref. 1)

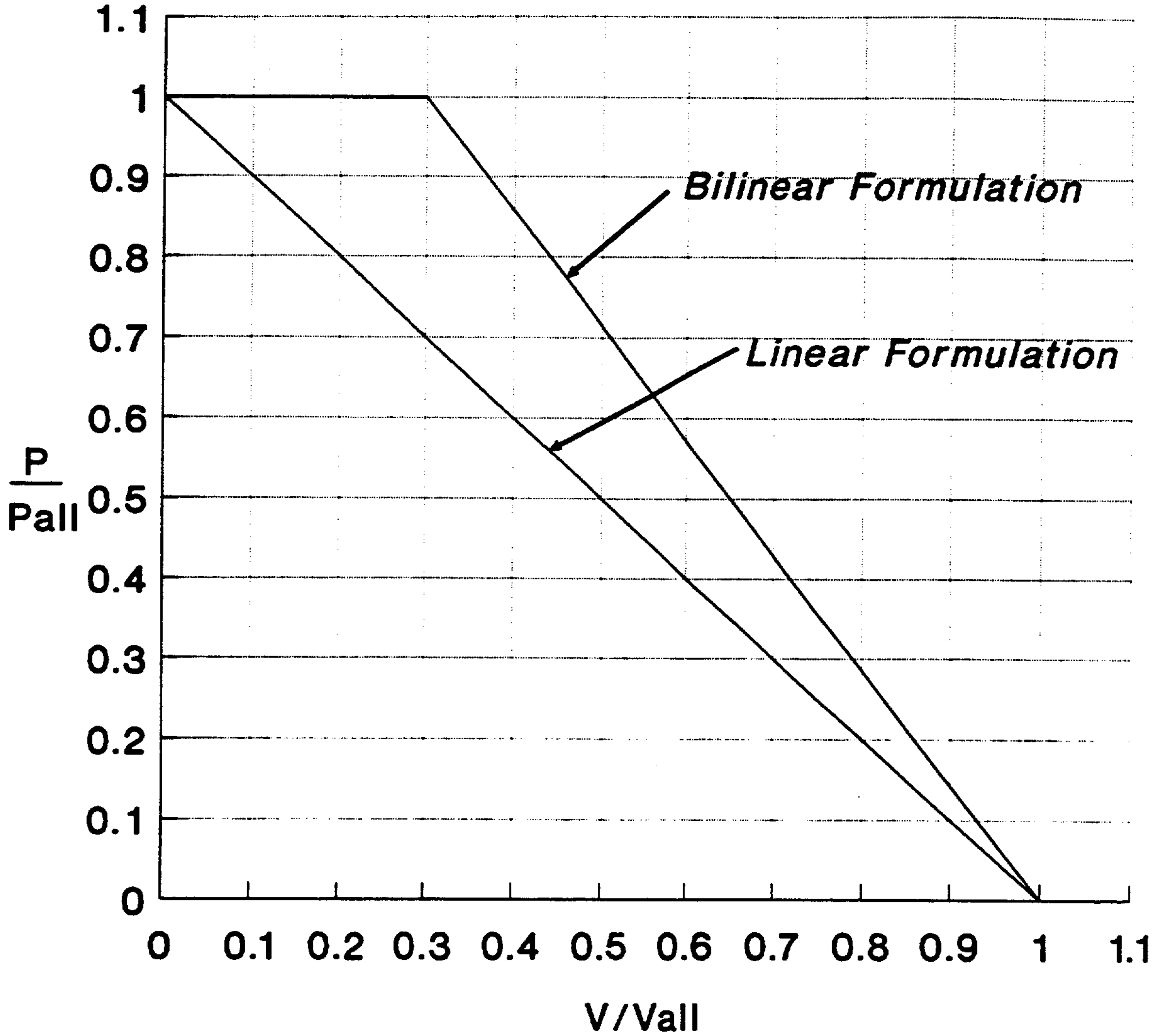
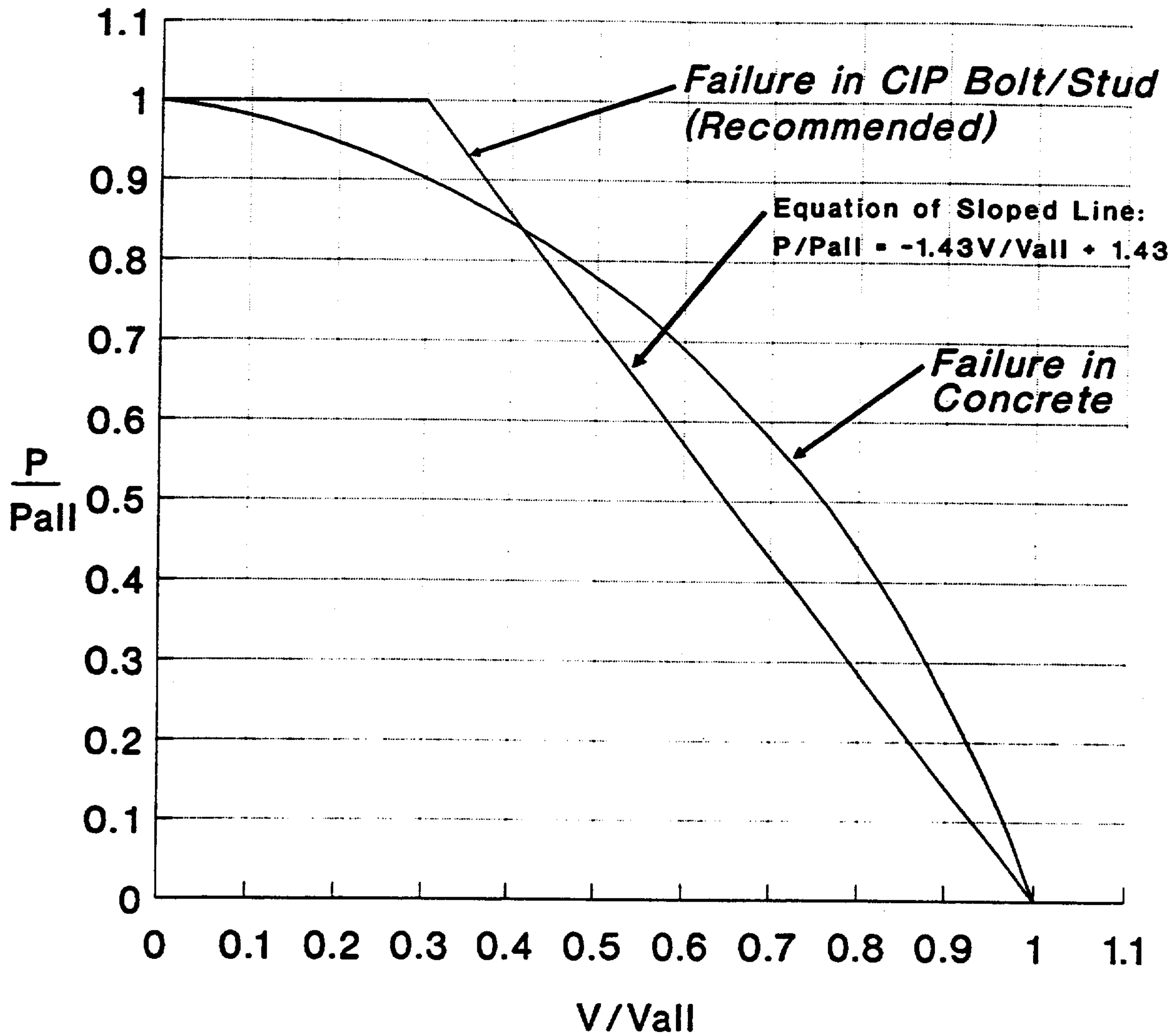


Figure 6.5-1 **Shear-Tension Interaction Limitations for Expansion Anchors (Reference 41) (Figure C.2-4 of SQUG GIP, Reference 1)**



V/Vall	0.0	0.1	0.3	0.5	0.7	0.9	1.0
P/Pall - CIP Bolt/Stud	1.0	1.0	1.0	.71	.43	.14	0.0
P/Pall - Concrete	1.0	.99	.91	.79	.60	.30	0.0

Figure 6.5-2 **Shear-Tension Interaction Limitations for Cast-In-Place Bolts and Headed Studs (Reference 41) (Figure C.3-2 of SQUG GIP, Reference 1)**

7. SEISMIC INTERACTION

7.1 INTRODUCTION[1]

The purpose of this section is to describe seismic interaction and techniques for evaluating its effects on equipment in DOE facilities. Seismic interaction is the physical interaction of any structures, piping, or equipment with a nearby item of equipment caused by relative motions from an earthquake. Components with fragile appendages (such as instrumentation tubing, air lines, and glass site tubes) are most prone to damage for seismic interaction. An inspection should be performed in the area adjacent to and surrounding equipment to identify any seismic interaction condition which could adversely affect the capability of the equipment to perform its intended function.

An overview of seismic interaction is shown in Figure 7.1-1. An earthquake can cause various types of interactions such as bumping, falling, or flooding. The SCEs should identify the various types of interactions and work with other SRT members to determine the overall effect on the facility. This chapter describes the seismic interaction effects covered by the DOE Seismic Evaluation Procedure and how they can be evaluated. The seismic interaction effects which are included within the scope of this procedure are proximity; structural failure and falling; flexibility of attached lines and cables and differential displacements; and water spray, flood, and fire hazards.

Using this chapter, the SCEs should be familiar with the various types of interaction, be able to judge if an interaction is credible and its significance during a walkdown, be able to identify outliers, and be familiar with DOE Guidance related to seismic interactions.

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1 Based on Section D.1 of SQUG GIP (Ref. 1)

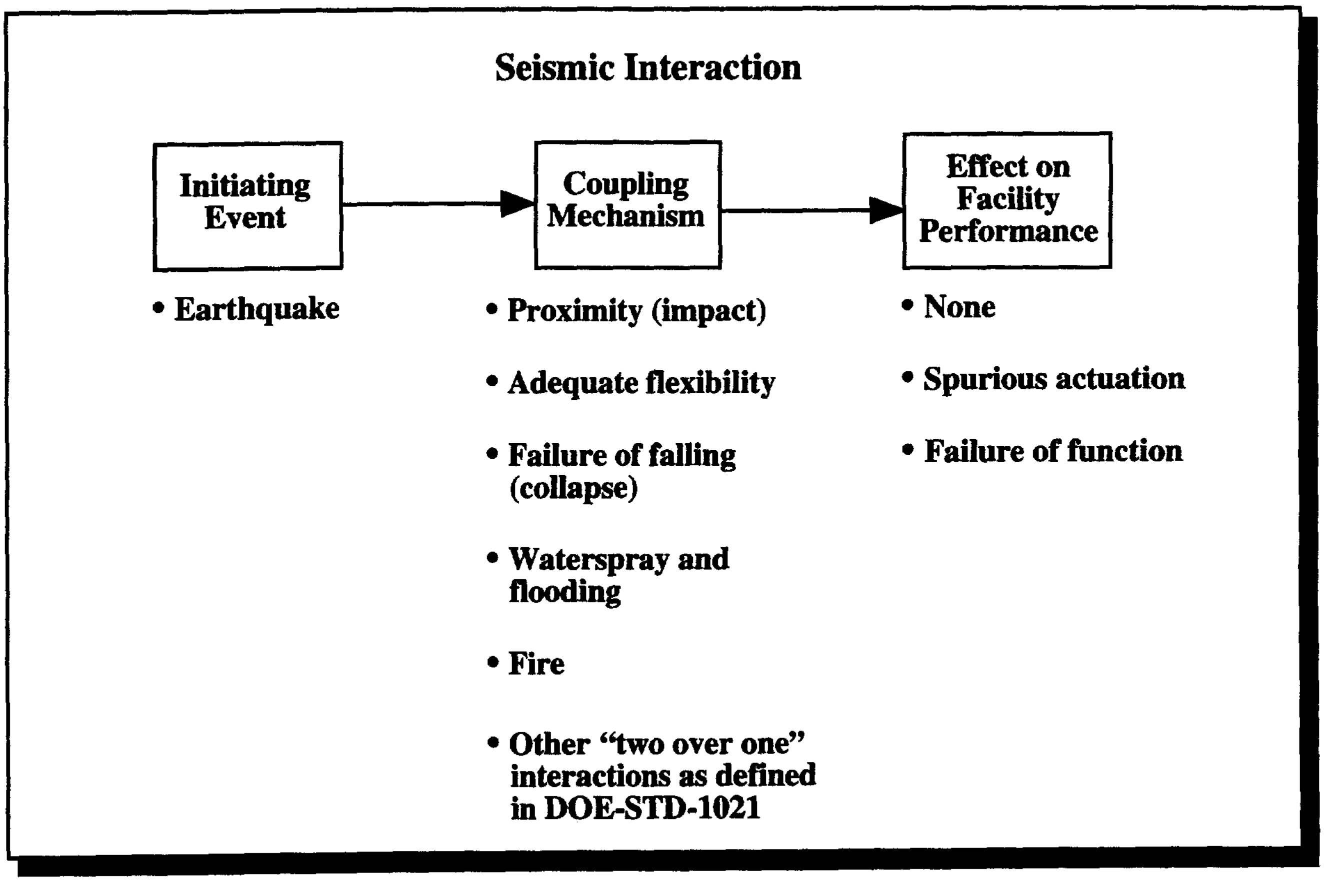

Figure 7.1-1 Overview of Seismic Interaction

7.2 INTERACTION EFFECTS

An example of the effects of seismic interaction is shown in Figure 7.2-1. The hanging conduit or piping, which is free to swing during earthquake motion, is the source, while the target is the electrical cabinet. The shaded zone in the figure defines the zone of influence where the source can affect the target. For a credible interaction to occur, the source must impact or interact with the target (see Figure 7.2-2). While evaluating the effects of credible seismic interactions, the SCEs must determine if the interactions are significant or not. The screening process for interaction effects includes evaluating the target, source, credibility, and significance. If all of these screens or considerations are satisfied, then the interaction being evaluated is an outlier and should be resolved as discussed in Chapter 12.

A significant interaction will compromise the intended performance and will affect the safety function of the equipment being evaluated. Examples of a significant interaction include an air-operated valve impacting a nearby structural column (see Figure 7.2-3), rupture of water sprinkler piping above medium-voltage switchgear, or a cart impacting a motor control center which contains vibration sensitive equipment such as essential relays.

A non-significant interaction, on the other hand, will not cause appreciable damage to the equipment being evaluated. Examples of a non-significant interaction include a light weight object impacting a large diameter conduit (see Figure 7.2-4) or small diameter piping impacting the outside casing of a rugged horizontal pump.

7.2.1 Proximity[2]

Seismic interaction due to proximity is the impact of adjacent equipment or structures on equipment due to their relative motion during seismic excitation. This relative motion can be the result of the vibration and movement of the equipment itself or any adjacent equipment or structures. When sufficient anchorage, bracing, adequate clearance, or other means are provided to preclude large deflections, seismic proximity effects are not typically a concern.

Even if there is impact between adjacent equipment or structures, there may not be any significant damage to the equipment. In such cases, this seismic interaction would not be considered a reason for concern, provided the equipment can still accomplish its intended function. One exception to this is electrical cabinets containing essential relays which are required to function. Since relays are susceptible to chatter, any impact on an electrical cabinet which has such an essential relay in it should be considered an unacceptable seismic interaction and cause for identifying that electrical cabinet as an outlier.

Examples of potential seismic interaction due to proximity include the following:

- Equipment carts, dollies, chains, air bottles, welding equipment, etc., may roll into, slide, overturn, or otherwise impact equipment

- Electrical cabinets that deflect and impact walls, structural members, another cabinet, etc., may damage devices in the cabinet or cause devices to trip or chatter

- Storage cabinets, office cabinets, files, bookcases, wall lockers, and medicine cabinets may fall or tip into equipment

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[2] Based on Sections D.2 and D.6 of SQUG GIP (Ref. 1)

- The doors on electrical cabinets may swing and impact devices or cause relays to chatter.
- Inadequately anchored or braced equipment such as pumps, vessels, tanks, heat exchangers, cabinets, and switchgear may deflect or overturn and impact equipment

The judgment of the SCEs should be used to differentiate between credible and non-credible interaction hazards.

7.2.1.1 Piping, Raceways, and Ductwork Deflections[3]

The motion of piping, conduit, cable raceways, and other distribution lines may result in impact interactions with equipment being reviewed. Non-safety-related piping is commonly supported with rod hangers or other forms of flexible dead load support, with little or no lateral restraint. Where adequate clearance with equipment is not provided, potential impact interaction may result. The integrity of the piping is typically not a concern. (Threaded fittings, cast iron pipes and fittings, and grooved type couplings may be exceptions where large anchor movement is possible.) In general, impacts between distribution systems (piping, conduit, ducts, raceways) and equipment of comparable size are not a cause for concern; the potential for large relative motions between dissimilar size systems should be carefully evaluated to assure that a large system cannot carry away a smaller one.

Judgment should be exercised by the SCEs in estimating potential motions of distribution systems in proximity to the equipment under evaluation. For screening purposes, a clearance of 2 inches for relatively rigid cable tray and conduit raceway systems and 6 inches for relatively flexible systems would normally be adequate to prevent impacts, subject to the judgment of the SCEs.

Where potential interaction may involve systems with significant thermal movements during facility normal operating conditions, the thermal displacements should be evaluated along with those resulting from seismic deflections. Inter-equipment displacement limits may be developed from the applicable floor response spectra to assist in this effort. In-structure response spectra (IRS) are discussed in Chapter 5.

7.2.1.2 Mechanical and Electrical Equipment Deflections[4]

Inadequately anchored or inadequately braced mechanical and electrical equipment, such as pumps, valves, vessels, cabinets, and switchgear, may deflect or overturn during seismic loading which results in impact with nearby equipment on the SEL. Certain items, such as tanks with high height-to-diameter aspect ratios, can deflect and impact nearby equipment. Electrical cabinets in proximity to each other may pound against each other or against walls and columns. Suspended equipment components such as room heaters and air conditioning units may impact with equipment.

The SCEs should use judgment in such cases to evaluate the potential displacements and their potential effect on nearby equipment being evaluated. Cabinets with essential relays warrant special concern.

7.2.2 Structural Failure and Falling[5]

Equipment listed on the SEL can be damaged and unable to accomplish its function due to impact caused by failure of overhead or adjacent equipment, systems, or structures. This interaction

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3 Based on Section D.2.1 of SQUG GIP (Ref. 1)

4 Based on Section D.2.2 of SQUG GIP (Ref. 1)

5 Based on Sections D.3 and D.6 of SQUG GIP (Ref. 1)

hazard is commonly referred to as a Category II over Category I concern. This seismic interaction effect can occur from nearby or overhead: (1) mechanical and electrical equipment; (2) piping, raceway, and HVAC systems; (3) architectural features; and (4) operations, maintenance, and safety equipment. The seismic interaction effects which are of concern for these types of equipment, systems, and structures are described below. It is the intent of this evaluation that realistic hazards be identified and corrected; failure of non-seismically supported equipment and systems located over equipment being evaluated should not be arbitrarily assumed.

Facility operations, safety, and maintenance equipment as well as facility architectural features are commonly overlooked in seismic design programs and present sources of seismic interaction concerns. Examples of potential seismic interaction due to failure and falling include the following:

- Partition walls and unreinforced masonry block walls
- Ceiling tiles on unrestrained T-bar grid systems
- Overhead walkway platform grating lacking tie-downs
- Suspended light fixtures and fluorescent tubes
- Storage cabinets, files, and bookcases
- Tool carts on wheels and tool chests
- Ladders and scaffolding
- Portable testing equipment
- Unrestrained gas bottles and fire extinguishers
- Unrestrained equipment on wall-mounted supports
- Unreinforced masonry walls adjacent to equipment may spall or fall and impact equipment or cause loss of support of equipment
- Emergency lighting units and batteries used for emergency lighting can fall or overturn and damage equipment by impact or spilling of acid
- Fire extinguishers may fall and impact or roll into equipment
- Intercom speakers can fall and impact equipment
- Cable trays, conduit systems, and HVAC systems, including HVAC louvers and diffusers, may fall and impact equipment
- Structures or structural elements may deform or fall and impact equipment
- Architectural features such as suspended ceilings, ceiling components such as T-bars and acoustical panels, light fixtures, fluorescent tubes, partition walls, and plate glass may deflect, overturn or break and fall and impact equipment
- Grating may slide or fall and impact equipment

The judgment of the SCEs should be used to differentiate between credible and non-credible interaction hazards.

7.2.2.1 Mechanical and Electrical Equipment[6]

Equipment such as tanks, heat exchangers, and electrical cabinets that are inadequately anchored or inadequately braced have historically overturned and/or slid due to earthquake excitation (see Figure 7.2-5). In some cases this has resulted in damage to nearby equipment or systems.

7.2.2.2 Piping, Raceways, and HVAC Systems[7]

Falling of non-seismically designed piping, raceways, and HVAC systems have been observed in very limited numbers during earthquakes. Most commonly reported are falling of inadequately secured louvers and diffusers on lightweight HVAC ducting. Damage to piping systems is less common and usually is limited to component failures which have rarely compromised system structural integrity. Typical damage is attributed to differential motions of systems resulting from movement of unanchored equipment, attachment of systems between buildings, or extremely flexible long runs of unrestrained piping. Very long runs of raceway systems pose a potential falling hazard when the runs are resting on, but not attached to, cantilever supports.

7.2.2.3 Architectural Features[8]

Architectural features include such items as ceilings, light fixtures, platform grating, unreinforced masonry walls, and other structures. The seismic interaction effects for these are described below:

- Ceilings. T-bar suspended tiles, recessed fixtures, and sheet rock are used in some facility areas (such as the control room). Seismic capabilities of these ceilings may be low. The SCEs should check for details that are known to lead to failure such as open hooks, no lateral wire bracing, etc. Section 10.5.2 discusses suspended ceilings.

- Light Fixtures. Normal and emergency light fixtures are used throughout the facility. Fixture designs and anchorage details vary widely. Light fixtures may possess a wide range of seismic capabilities. Pendant-hung fluorescent fixtures and tubes pose the highest risk of failure and damage to sensitive equipment. The SCEs should check for positive anchorage, such as closed hooks and properly twisted wires. Typically this problem is not caused by lack of strength; it is usually due to poor connections. Emergency lighting units and batteries can fall and damage equipment being reviewed due to impact or spillage of acid.

- Platform Gratings. Unrestrained platform gratings and similar personnel access provisions may pose hazards to impact-sensitive equipment or components mounted on them. Some reasonable positive attachment is necessary, if the item can fall.

- Unreinforced Masonry Walls. Unreinforced, masonry block walls should be evaluated for possible failure and potential seismic interaction with equipment being reviewed unless the wall has been seismically qualified. The SCEs should review the documentation for masonry walls to determine which walls have and which walls have not been seismically qualified. Section 10.5.1 discusses the qualification of these types of walls.

- Structures. If equipment being reviewed is located in lower Performance Category structures, then potential structural vulnerabilities of the building should be identified; however, facility structures are typically seismically adequate.

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[6] Based on Section D.3.1 of SQUG GIP (Ref. 1)
[7] Based on Section D.3.2 of SQUG GIP (Ref. 1)
[8] Based on Section D.3.3 of SQUG GIP (Ref. 1)

7.2.2.4 Operations, Maintenance, and Safety Equipment[9]

Facility operations and maintenance require specialized equipment, some of which may be permanently located or stored in locations near safety systems. Some operations, maintenance, and safety equipment is designed so that it may be easily relocated by facility personnel. Where equipment design or facility operating procedures do not consider anchorage for permanently located equipment, this equipment may slide, fall, overturn, or impact with equipment listed on the SEL. Typically such equipment includes:

- Cabinets and Lockers. Inadequately restrained floor and wall-mounted filing cabinets and equipment storage lockers may result in overturning or falling and impact.

- Gas Storage Bottles. Unrestrained or inadequately restrained gas bottles may result in overturning and/or rolling and this may cause impact. In addition, the gas bottles can become high velocity projectiles if the reducing valve is snapped off and the gas bottles overturn and/or roll. Section 10.3.2 discusses further considerations for gas bottles.

- Refueling Equipment. Refueling equipment such as lifting equipment and servicing and refueling tools may be stored in proximity to equipment being evaluated. Inadequately restrained equipment may pose hazards.

- Monorails, Hoists, and Cranes. Monorails and service cranes are permanently located over heavy equipment requiring movement for service. Falling of service crane components such as tool and equipment boxes may result from inadequate component anchorage. They should be restrained from falling. Judgment by the SCEs should be used to assess the potential for and consequences of such equipment falling.

- Radiation Shields, Fire Protection and Miscellaneous Equipment. Temporary and permanent radiation shielding may pose hazards. Miscellaneous maintenance tools, such as chains and dollies, test equipment, fire protection equipment, fire extinguishers, and hose reels may fall if inadequately restrained. Equipment carts may roll into equipment being evaluated.

7.2.3 Flexibility of Attached Lines and Differential Displacements[10]

Distribution lines, such as small bore piping, tubing, conduit, or cable, which are connected to equipment can potentially fail if there is insufficient flexibility to accommodate relative motion between the equipment and the adjacent equipment or structures. Straight, in-line connections in particular are prone to failure. The scope of review for flexibility of these lines extends from the item of equipment being evaluated to their supports on the building or nearby structure. In addition, the review should consider operational concerns for the lines, such as the relationship of the lines to any check valve and sources of supply for the lines.

Distribution systems that span between different structural systems need to have sufficient flexibility to accommodate differential motion of the supporting structures (see Figure 7.2-6). Piping may be vulnerable where it interfaces with a building structure foundation.

[9] Based on Section D.3.4 of SQUG GIP (Ref. 1)

[10] Based on Sections D.4 and D.6 of SQUG GIP (Ref. 1)

Examples of potential seismic interaction due to flexibility of attached lines include the following:

- Piping, cable trays, conduit, and HVAC may deflect and impact equipment

- Anchor movement may cause breaks in piping, cable trays, conduit, HVAC, etc. which may fall or deflect and impact adjacent equipment

The judgment of the SCEs should be used to differentiate between credible and non-credible interaction hazards.

7.2.4 Water Spray, Flood, and Fire Hazards

Potential seismic-induced spray, flood, and fire interaction sources should be evaluated and a few examples include the following:

- Hazardous/flammable material stored in unanchored drums, unanchored shelves, or unlocked cabinets

- Nonductile fluid-carrying pipe (such as cast-iron or PVC pipe) (see Figure 7.2-7)

- Fire protection piping with inadequate clearance around fusible-link sprinkler heads (see Figure 7.2-8)

- Natural gas lines and their attachment to equipment or buildings

- Acetylene bottles

- Mechanical and threaded piping couplings can fail and lead to pipe deflection or falling and impact on equipment. Grooved type couplings used in fire protection piping are one example of this type of mechanical coupling

- Sheetrock may fall and impact equipment if it was previously water-damaged or if there is severe distortion of the building

- Unanchored room heaters, air conditioning units, sinks, and water fountains may fall or slide into equipment

The judgment of the SCEs should be used to differentiate between credible and non-credible interaction hazards.

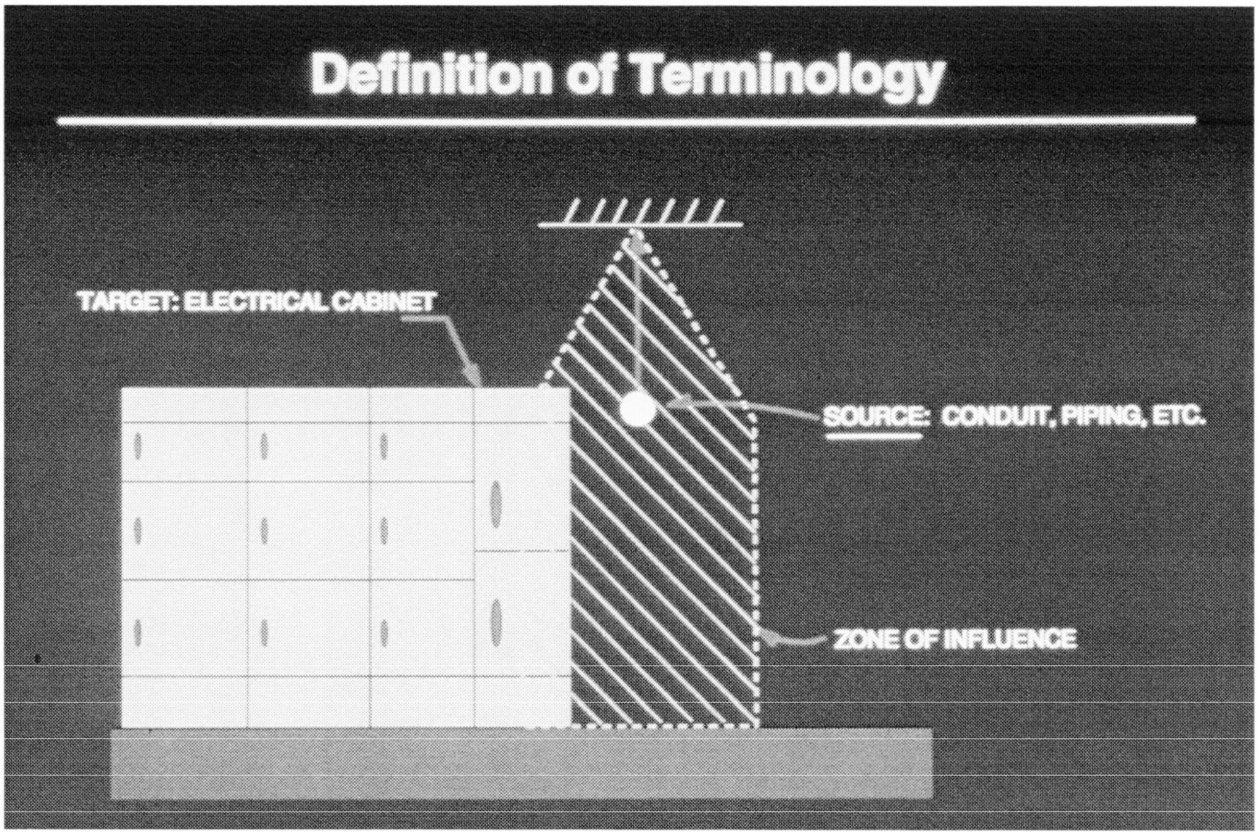

Figure 7.2-1 Example of Seismic Interaction

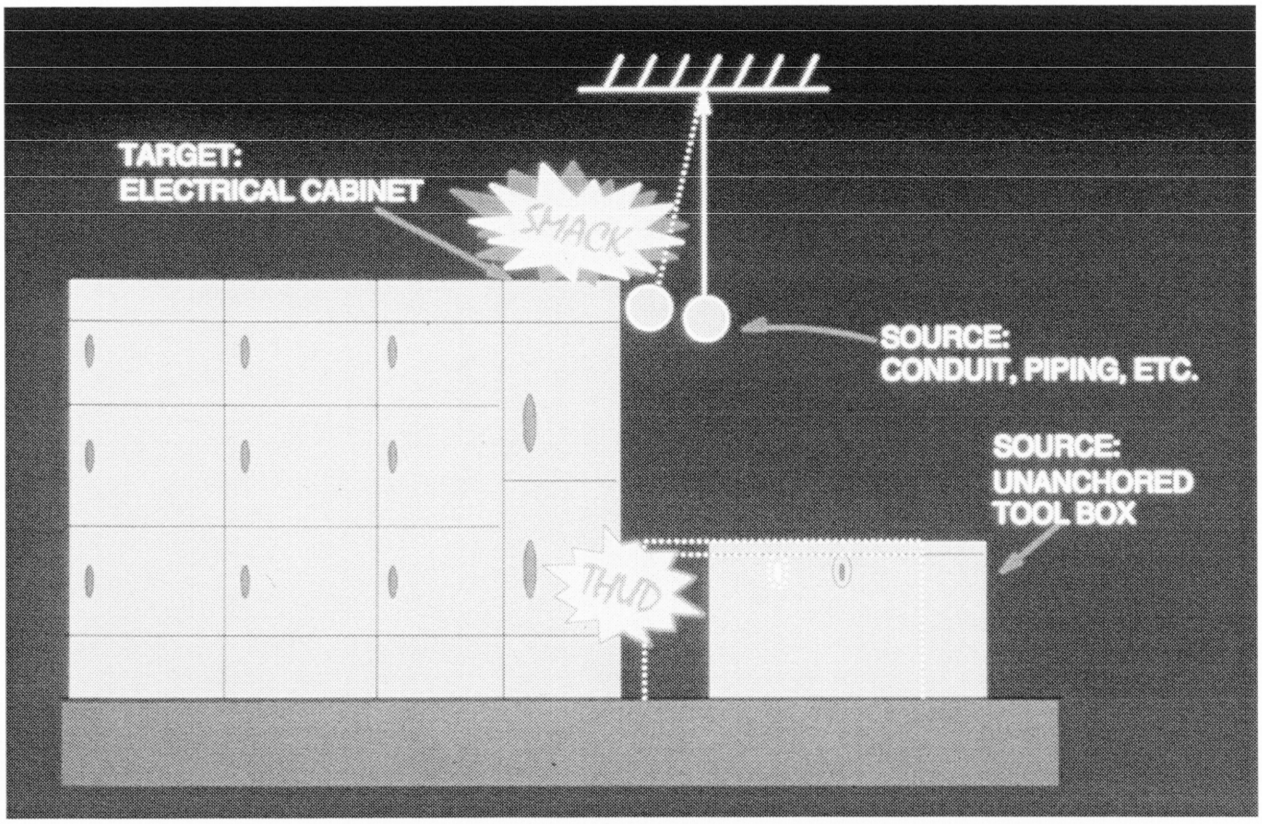

Figure 7.2-2 Example of Credible Interactions

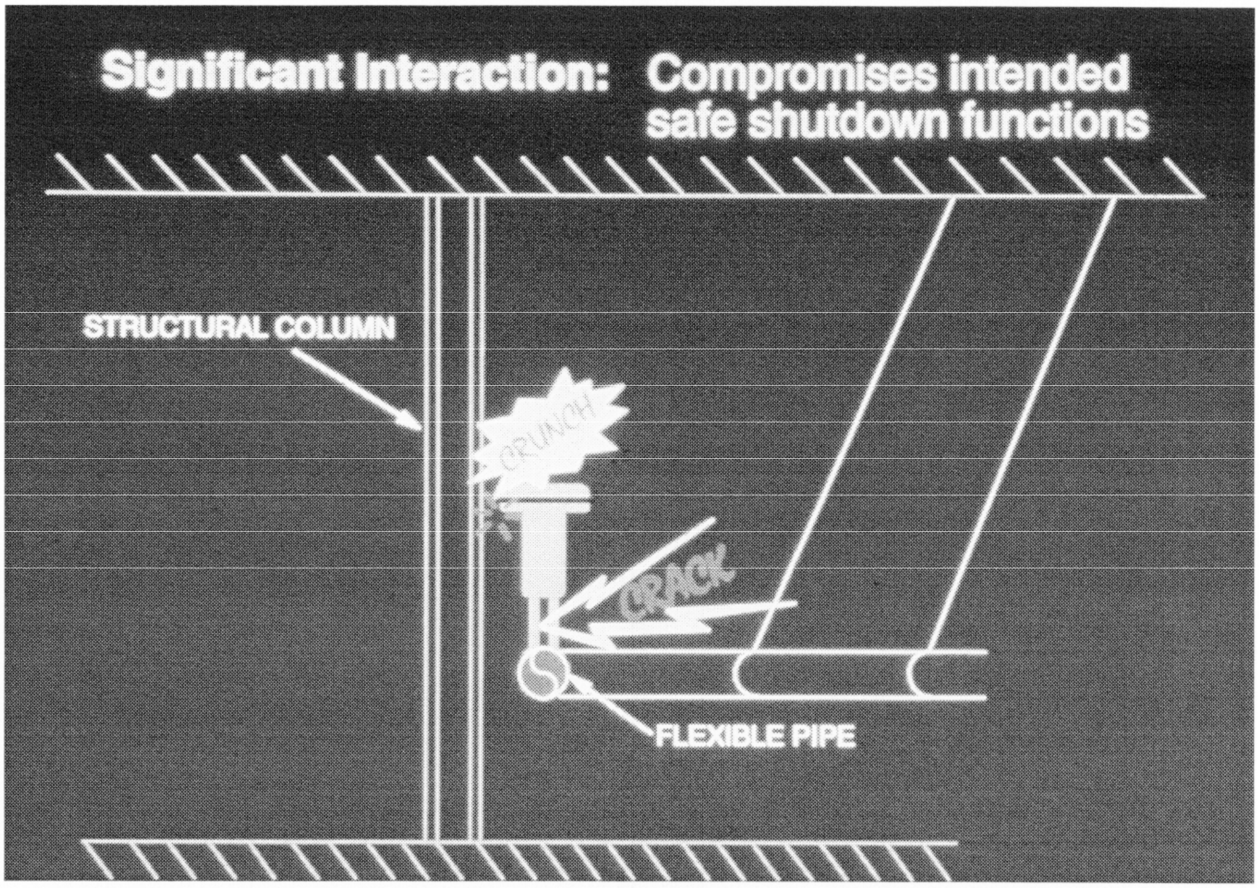

Figure 7.2-3 **Example of Significant Interaction which Compromises Intended Safety Functions**

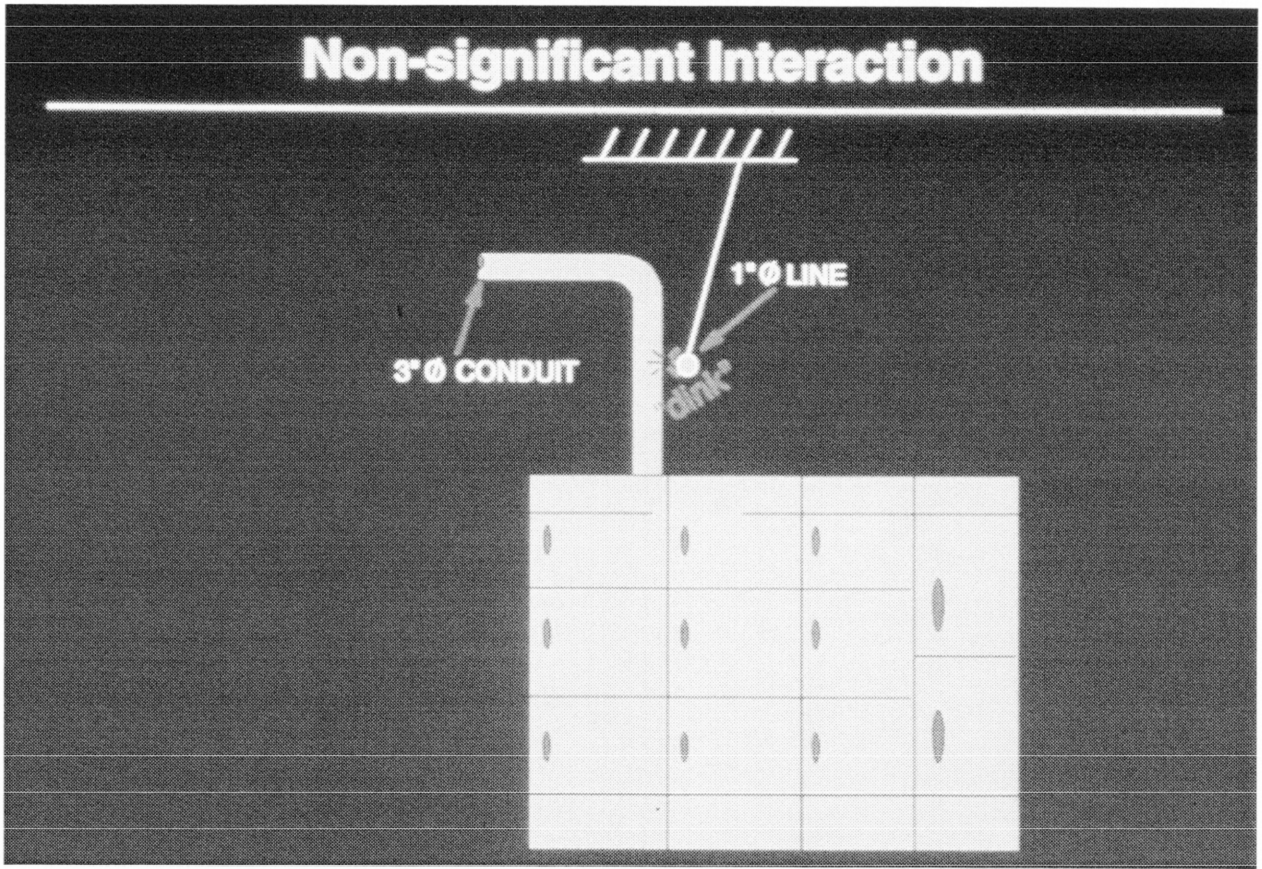

Figure 7.2-4 Example of Non-Significant Interaction

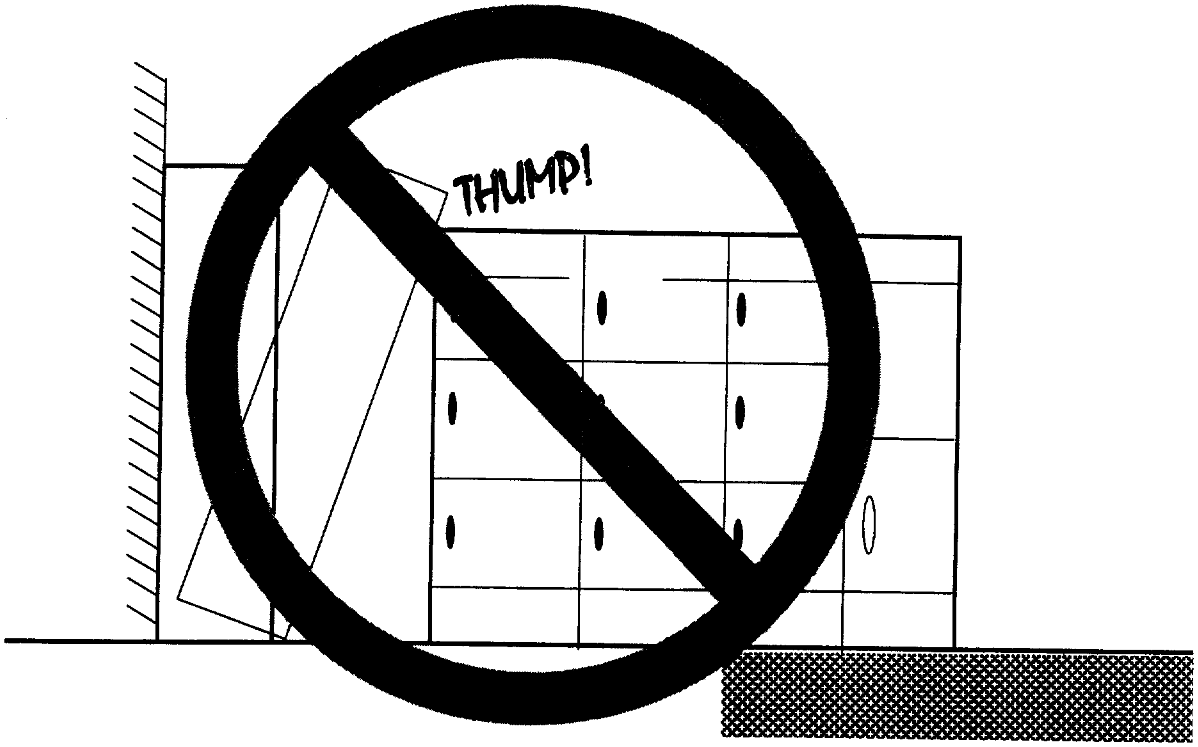

Figure 7.2-5 Failure and Falling Interaction Hazards

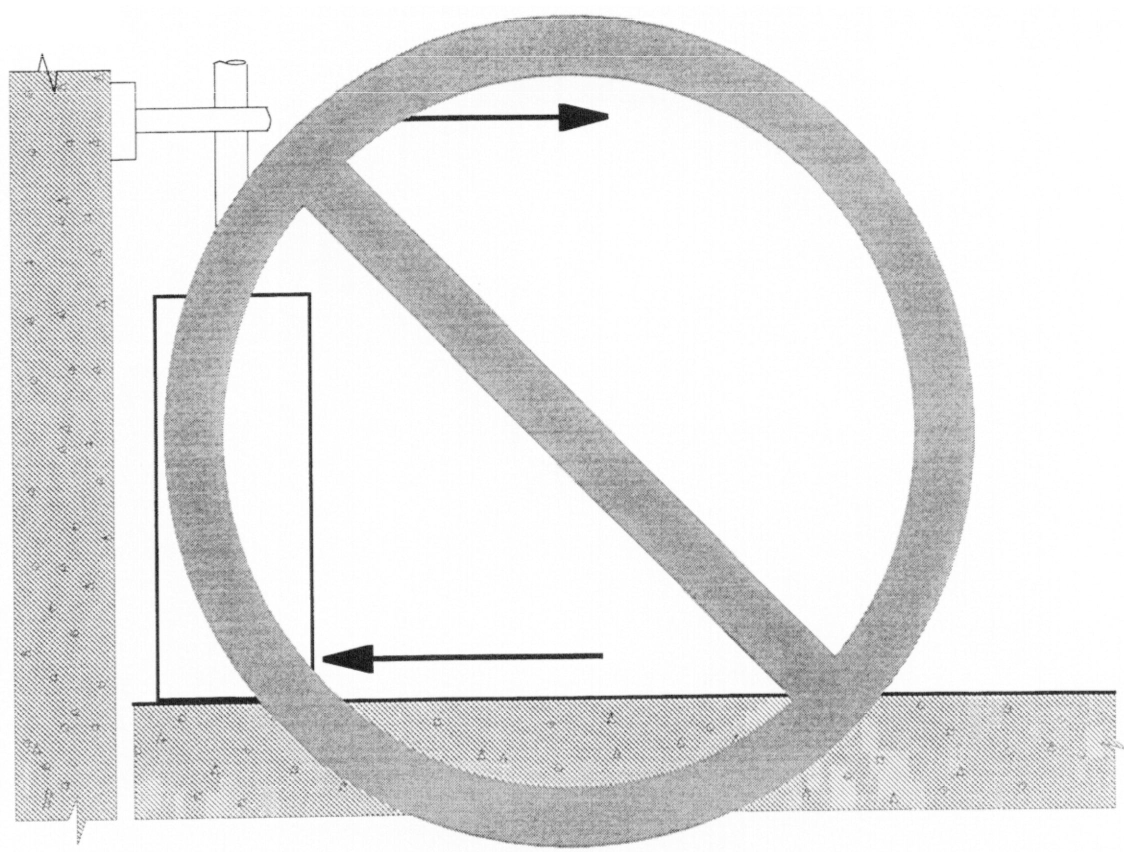

Figure 7.2-6 Differential Displacement Interaction

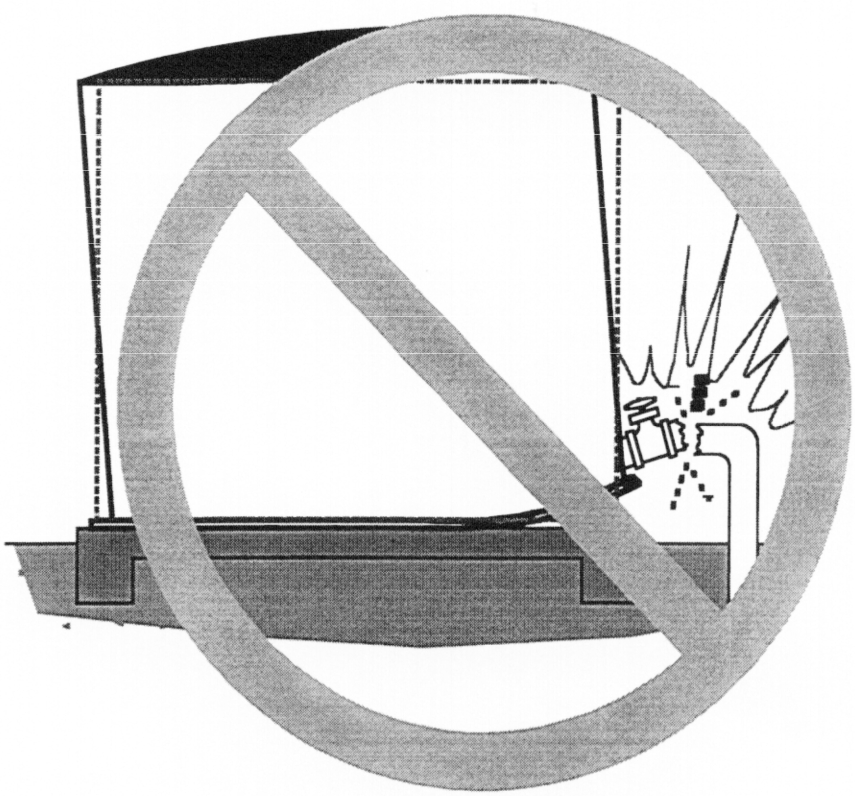

Figure 7.2-7 Pipe Break Potential for Unanchored Tanks

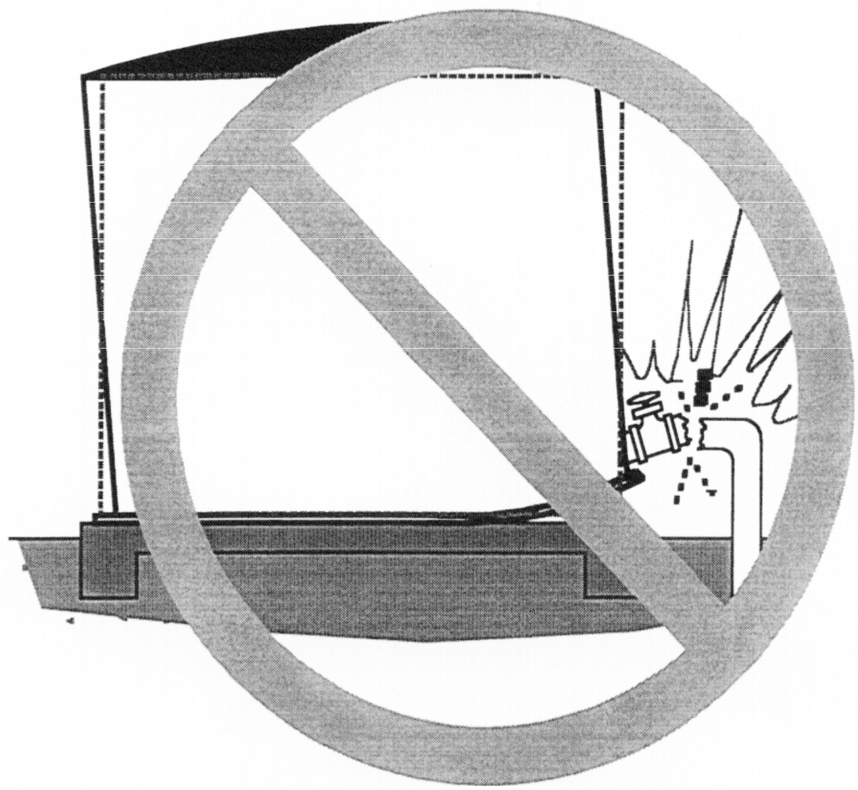

Figure 7.2-7 Pipe Break Potential for Unanchored Tanks

7.3 DOE GUIDANCE

Guidance on the treatment of seismic interaction effects is included in DOE-STD-1021, "Natural Phenomena Hazards Performance Categorization Guidelines for Structures, Systems, and Components" (Ref. 7). This guidance focuses on "two over one" concerns and should be used to evaluate the seismic interaction effects discussed in Section 7.2. "Two over one" concerns, as discussed in DOE-STD-1021 and DOE-STD-3009 (Ref. 11), are those with a lower safety class structure, system, or component (SSC) located above, or able to interact with, a higher safety class SSC. Further detailed information on selecting performance and hazard categories is provided in References 7, 10, and 11.

7.3.1 System Interaction Effects[11]

(a) An SSC that has been preliminary categorized in accordance with the basic performance categorization (PC) guidelines of Section 2.4 of Reference 7 (the source) shall have appropriate additional seismic mitigation requirements as provided in Paragraphs (b), (c) and (d) below, if its behavior by itself, or the multiple common-cause behavior of it with other SSCs may adversely affect the performance of other SSC (the target). These additional requirements will depend on the type of source behavior that causes adverse interaction with the target during or following an seismic event.

(b) If the source behavior that causes adverse interaction is within the acceptable behavior limits of the source (i.e., if the adverse interaction occurs before failure) adequate measures shall be taken to preclude such interaction and to ensure that the performance goal for the target is preserved. For example, assume that the postulated seismic deflection of a performance category (PC)-1 cabinet (source) is within its own acceptable behavior limits, but the cabinet can potentially impact and fail a PC-2 fire-suppression component (target). To prevent this adverse interaction, the cabinet support system or the cabinet itself can be stiffened/strengthened in such a way that the calculated deflection of the cabinet towards the target, when subjected to a seismic level corresponding to the performance category of the target, is less than the available clearance by a factor equal to the applicable design margin for the target. Alternatively, a barrier can be provided to preclude the adverse interaction and to protect the target. Such a barrier shall be designed to withstand seismic effects combined with the interaction effects from the source (in this case the impact from the PC-1 cabinet). To ensure that the target performance goal is preserved, the barrier shall be placed in the same performance category as the target (in this case PC-2).

(c) If the adverse interaction is possible only after the source fails or exceeds its acceptable behavior limits, either of the following two requirements shall be met to preclude adverse interaction:

  (i) The source shall have additional seismic requirements corresponding to the performance category of the target, if the failure probability of the target, given the failure of the source, is greater than one percent. If the implementation of this criteria is judged not to be cost-effective, the additional seismic mitigation requirements for the source shall be in accordance with Table 7.3-1. In either case, these additional requirements can be restricted to the source failure mode related to the adverse interaction effects.

  (ii) Adequate measures shall be taken to preclude adverse interaction and to ensure that the performance goal for the target is preserved. Examples of acceptable measures

[11] Based on Section 2.5 of DOE-STD-1021 (Ref. 7)

Table 7.3-1 System Interaction Effects on Performance Categorization (Reference 7)

Performance Category of Target SSC(1)	Preliminary Performance Category of Source SSC(2)	Range or Limit of Target Failure Probability Due to Interaction(3) (p)	Revised NPH Requirements of Source SSC(4)
PC-4	PC-3	$p > 10\%$	PC-4
		$p \leq 10\%$	PC-3(5)
	PC-2	$p > 10\%$	PC-4
		$1\% < p \leq 10\%$	PC-3
		$p \leq 1\%$	PC-2(5)
	PC-1	$p > 10\%$	PC-4
		$1\% < p \leq 10\%$	PC-3
		$p \leq 1\%$	PC-1(5)
PC-3	PC-2	$p > 10\%$	PC-3
		$p \leq 10\%$	PC-2(5)
	PC-1	$p > 10\%$	PC-3
		$p \leq 10\%$	PC-1(5)
PC-2	PC-1	$p > 10\%$	PC-2
		$p \leq 10\%$	PC-1(5)

SSC - Structure, System, or Component
NPH - Natural Phenomena Hazard
PC - Performance Category

Notes:

(1) If the target consists of more than one SSC, the highest performance category of the group shall be considered here.

(2) This is the preliminary performance category of the source SSC before considering system interaction effects. Note that PC-0 is not considered here because a PC-0 SSC cannot have any adverse effect on the performance of PC-1 through PC-4 SSCs

(3) This is the approximate probability of exceedance of acceptable behavior limit for the target SSC given that the source SSC will fail and interact with target SSC due to NPH effects.

Thus, if the target is a PC-4 SSC that may be adversely affected by the failure of a PC-2 SSC (source), and if the target failure probability due to this interaction is greater than 10%, then one of the methods of precluding the interaction will be to subject the source to additional NPH requirements corresponding to PC-4 (see also note 4 below).

(4) The source SSC shall be designed/evaluated to those requirements of the revised performance category that are essential for precluding adverse interaction with the target (in other words, it is not necessary to satisfy the functional requirements of the source SSC when subjected to these additional NPH requirements unless essential for precluding adverse interaction).

The basis for determining the revised NPH requirements for the source SSC is that the performance goal of the target SSC shall not be compromised because of system interaction effects, i.e. the product of the performance goal for the revised source performance category and the target failure probability must not be more than the performance goal of the target SSC. However, to account for uncertainties in determining target failure probabilities, the limiting values in the 3rd column of the table have been selected conservatively (i.e. lower than the values computed on the above basis).

(5) For these cases, consideration of interaction effects does not require additional NPH requirements for the source SSC.

are: stiffening/strengthening of the source structure or support system, relocating the source and/or the target, installing barriers, installing new components, modifying existing components, or any combination of these measures.

(d) If the behavior or failure of a source can adversely affect the performance of more than one target, the source shall have additional seismic requirements corresponding to the highest performance category that is determined by applying the rules provided in Paragraphs (a), (b), and (c) above separately for each target.

7.3.2 Determination of System-Interaction-Related Target Failure Probability[12]

To account for adverse system interaction, the determination of failure probability of the target component given the failure of the source component is required. Depending on the physical and functional complexity of the target and the nature of its interaction with the source, the level of effort in determining this target failure probability can vary. Following the" graded approach" philosophy, the level of rigor with which such failure probabilities are to be determined should depend on the safety significance and the preliminary performance category of the target, the hazard category of the facility, and the relative cost of various methods of determining target failure probabilities.

In the following paragraphs two methods of determining or estimating target failure probabilities are presented in order of decreasing rigor.

(a) Systematic Analysis Method

Target failure probabilities can be determined using a systematic analysis approach by constructing a fault-tree of the scenario. If justifiable from cost-benefit considerations, this may be a desirable method when necessary data is available. Generally, it should be used when the failure of the target is dependent on a large and complex chain of events that may follow the failure of the source, or to qualify a large system in its entirety. Component-by-component application of this method is unlikely to be cost-effective.

(b) Approximate Method

In this method, the effects of source failure on target are modeled approximately, but rationally, considering possible scenarios identified by review of system design. Even though such models are approximate, their analyses provide good "order-of magnitude" type of data that are often adequate for the purpose. Examples of the use of this method are given in Section 7.3.4.

7.3.3 Application of System Interaction Rules[13]

The consideration of adverse effects of system interaction of one component or system (source) on the other (target) is very important in determining performance categories of SSCs. Adverse interaction effects can be different for different systems. Examples of common adverse interaction effects are:

(i) Structural Failure and Falling (see Section 7.2.2): Inadequately designed, inadequately anchored, and unanchored components may fail, slide, and/or topple and fall on or bump into other components that are not designed to withstand such interaction effects.

[12] Based on Section 3.8 of DOE-STD-1021 (Ref. 7)
[13] Based on Section 3.9 of DOE-STD-1021 (Ref. 7)

(ii) Proximity and Impact (see Section 7.2.1): Adjacent components may impact each other causing damage if the clearance between them is inadequate for seismic - induced deflections. Such adverse interaction may occur even if the deflection of the source is within its design limits.

(iii) Differential Displacement (see Section 7.2.3): A target distribution system (e.g., vital cable trays, pipes, ventilation ducts) may span between different structural systems (source). Differential displacement may be within acceptable behavior limits for the individual structures, but may still affect the distribution systems adversely.

(iv) Mechanical or Electrical Failure (see Section 7.2.4): The failure of a source mechanical or electrical component may impair the safety function of another component or system (e.g., the failure of a valve in a non-safety water distribution system causing flooding that short-circuits a safety class electrical motor).

Paragraph (b) of Section 7.3.1 provides the general requirements for precluding interaction that can occur before the source fails or reaches its acceptable behavior limits. Paragraph (c) of Section 7.3.1 provides three options to meet the requirements for precluding adverse interaction that can occur only when the source fails. The following paragraphs provide additional discussions on these three options:

(a) The first guideline in Paragraph (c)(i) of Section 7.3.1 is the most conservative of the three options, because it requires additional seismic requirements if the failure probability of the target exceeds only 1%. But it can also be most costly, since it may require upgrading the SSC. Hence, this guideline should be used when:

    (i) upgrading of the source does not involve a significant design change, or

    (ii) the existing source design has an adequate margin to withstand the same seismic level as the target.

(b) The second option in Paragraph (c)(i) of Section 7.3.1 requires the determination of target failure probability values, and depending on these values, the source may or may not need to be subjected to additional seismic requirements (see Table 7.3-1).

    This guideline should be used if the application of conservative "one-percent" rule cannot be justified from cost-benefit considerations. For example, if it is determined that the application of the "one percent" rule will require a PC-1 source to have seismic requirements equivalent to a PC-4 SSC resulting in expensive design changes, the use of Table 7.3-1 should be considered to reduce unnecessary conservatism.

(c) The third option given in Paragraph (c)(ii) of Section 7.3.1 requires the use of a barrier to prevent the source from interacting with the potential target. Very often this can be the most practical and cost-effective option. The barrier must be placed in the same performance category as the target, and be designed to withstand the interaction effects from the source in addition to the seismic loads.

7.3.4 Examples of Categorization Using System Interaction Rules[14]

This subsection provides few examples of the application of categorization rules considering system interaction effects as provided in Section 7.3.1.

(a) Example 1

Consider an emergency diesel generator in a Hazard Category 2 facility that is classified as a safety system using appropriate DOE orders and general design criteria. The diesel, generator, and all their support systems (e.g., fuel, lubrication, cooling water, and DC power systems) that perform a safety function should be evaluated as PC-3 in accordance with the provisions of Section 2 of Reference 7.

Consider the fluorescent light (source) hung directly above the diesel. For this case, assume that the light is not needed for required operator actions following a seismic event. Hence its preliminary performance category is PC-1. Diesels themselves are fairly rugged, and a falling lightweight object, like the light fixture is unlikely to damage them. However, there are some possible weak spots, particularly in the peripheral support systems (e.g., lubrication lines) that might be damaged and result in system failure. Assume that, in accordance with Section 3.8 of Reference 7, the failure probability of the diesel resulting from the falling light fixture is estimated to be approximately 25%. (This probability assumes the lighting fixture will fall. No credit is given at this stage for its design.) Following Paragraph (c)(ii) of Section 7.3.1, the lightning fixture should then be placed in PC-3.

(b) Example 2

Consider a case in which batteries for an uninterruptible power supply (UPS) in a Hazard Category 3 facility are in the same room with a 2000-gallon water storage tank. The UPS is classified as a safety system but the water storage tank is not. The UPS batteries (and their rack, connections, and the surrounding room structure) should be evaluated as PC-2 in accordance with the provisions of Section 2 of Reference 7.

Initially, the water storage tank might be considered as PC-1 (i.e., preliminary performance category). However, a systems-interaction check discloses that UPS batteries will short out during water immersion if only 1000 gallons of water flood the room. Thus, in accordance with criterion given in Paragraph (a)(i) of Section 7.3.1, the 2000-gallon tank should have the same performance category as the UPS batteries, that is, PC-2.

But what if the water was stored instead in ten 200-gallon tanks? The individual failure of each tank would not fail the UPS. However, if "multiple common-cause failure" is considered, one could reason that all ten tanks would be affected in the same way by the seismic event and simultaneous failure of several tanks might occur, leading to flooding of the batteries. Thus, each 200-gallon tank should also be placed in PC-2 in accordance with the provisions of Section 2 of Reference 7.

(c) Example 3

Consider a 100-foot-tall smoke stack for a laundry building at a DOE site that is not part of any safety system. However, its failure (from winds or earthquakes) would be costly and could injure workers, so initially it would be classified as Preliminary PC-1. Consider that there is a single Hazard Category (HC) 3 safety system component (say a PC-2 outside pump) that is 90 feet from

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[14] Based on Section 3.10 of DOE-STD-1021 (Ref. 7)

the base of the stack. A systems interaction analysis may assume that the stack would fall in essentially one piece and would fail the pump if it hits it. But the stack is equally probable to fall in any radial direction and the target size of the pump is small, fitting into a 2 degree angle. It is concluded that the probability of the stack hitting the component is less than 1%. Thus in accordance with Paragraph c(ii) of Section 7.3.1, the stack can be retained in PC-1.

(d) Example 4

Consider a Hazard Category 1 facility that relies heavily on operator actions, rather than seismically-qualified instrumentation and automatic control systems, to maintain a safe-state following a design basis earthquake. According to Section 2 of Reference 7, safety system SSCs of this facility should be placed in PC-4. In addition, SSCs needed to permit required operator actions following a design basis earthquake must also be classified as PC-4.

As an example, assume that one earthquake procedure written for this facility requires that an operator would go inside the pump room to read a water level gauge (which is seismically-qualified), and then relay this information to the control room via a system of walkie-talkies (assume that inside telephone lines are not seismically qualified). Items needed to permit this action, and thus which must meet PC-4 criteria, include all access doors (deformation of the door frames may be critical), emergency lightning and communication systems (the storage of flashlights and walkie-talkies could become a seismic design consideration), and any water or steam line whose seismic failure would be hazardous to the operator.

7.4 EVALUATION OF INTERACTION EFFECTS[15]

The SCEs should identify and evaluate all credible and significant interactions in the immediate vicinity of the equipment listed on the SEL. This includes consideration of seismic interactions on the equipment itself and on any connected distribution lines (e.g., instrument air lines, electrical cable, and instrumentation cabling) which are in the vicinity of the item of equipment. Evaluation of interaction effects should consider detrimental effects on the capability of equipment and systems to function; taking into account equipment attributes such as mass, size, support configuration, and material hardness in conjunction with the physical relationships of interacting equipment, systems, and structures. In the evaluation of proximity effects and overhead or adjacent equipment failure and interactions, the effects of intervening structures and equipment which would preclude impact should be considered. The effects of fire, flooding or exposure to fluids from ruptured vessels and piping should also be examined.

As summarized in this chapter, the considerations for seismic interaction effects include the following:

1. Soft targets free from impact by nearby equipment or structures.

2. If equipment contains sensitive essential relays, equipment free from all impact by nearby equipment or structures.

3. Attached lines have adequate flexibility.

4. No collapse of overhead equipment, distribution systems, or masonry walls.

5. Equipment is free from credible and significant seismic-induced flood and spray concerns.

[15] Based on Section D.5 of SQUG GIP (Ref. 1)

6. No credible seismic-induced fire concerns.

7. No other "two over one" concerns as defined in DOE-STD-1021.

8. No other concerns.

Good housekeeping within a facility can prevent many possible sources of seismic interaction. Miscellaneous equipment or supplies such as carts, ladders, brooms, and dollies can be easily stored such that they do not become sources of seismic interaction. In addition, the general arrangement of the facility and its contents can be developed to accommodate clearances and "stay-out" zones for the equipment being evaluated.

Damage from interaction in earthquakes is from unusual circumstances or from generic, simple details such as open hooks on suspended lights. The SCEs should spend most of their time evaluating: 1) unusual impact situations, and 2) lack of proper anchorage or bracing. The SCEs should not be concerned much with interaction issues due to piping and other system or structural component failures.